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FISH/CULVERT

ROADWAY DRAINAGE GUIDE

U.S. DEPARTMENT OF AGRICULTURE,
FOREST SERVICE, ALASKA REGION

Draft of Series No. R-10-42, September 1978

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REVIEW DRAFT--NOT FOR PUBLICATION

FISH / CULVERT
ROADWAY DRAINAGE GUIDE

This guide is designed as an aid to engineers, biologists, and hydrologists to help solve the problem of providing fish passages through drainage structures. Continued revision of this text and subsequent application will help the Forest Service carry out its responsibility of insuring that the installation of drainage structures or their operation in streams on national forest lands will not interfer with the passages of fish.

Useful material has been selected from several reference sources and combined to form a single source of information for the Alaska Region. Adult or juvenile salmonids or both are found in countless stream and tributary systems throughout the region. Their free passage and migration in these streams must be ensured.

Coho, steelhead, cutthroat, and Dolly Varden tend to spawn in headwater areas, and their fry disperse downstream. Juvenile fingerlings move about considerably as rearing populations adjust themselves to carrying capacities of their environments.

Juveniles also move upstream in significant numbers to winter in small tributaries where temperatures are moderated by groundwater sources. Roadcrossing structures, such as round culverts, when they are improperly designed and installed may cause increases in water velocity resulting in scouring at the downstream ends during periods of high water runoff. The scouring of gravel below culverts results in streambed instability and in culvert outlets becoming elevated above the normal water levels.

Since the jumping ability of juvenile salmonids is limited and their swimming capabilities in high-velocity currents are restricted, fishery biologists recommend the use of crossing structures which maintain the natural stream gradients, widths, and bottom materials. These requirements are generally best met by using small bridges or open-bottom arch culverts.

A table has been developed listing various types of drainage structures that are to be used to pass fish (Table 1). The delineations are noted as acceptable practices with the intent of providing maximum flexibility. The IDT approach to making environmental assessments results in recommended management presciptions that will identify the particular type of structure to be used in each case. This procedure will be coordinated with other Governmental agencies.

Representative drawings have been prepared showing the various methods described in the table. These are example drawings only and each method proposed will have to be designed for the specific site. Normally, culvert design is not compatible with fish passage because the pipe is smoother than the natural stream channel causing an increase in velocity.

When the pressure head and velocity head in a culvert are greater at the discharge end than the stream supported naturally, the resulting sudden expansion of water at the outlet causing digging and the creation of a pool.

For rearing streams, culverts should be installed at near zero gradient and with a bed roughness equal to the natural stream bed roughness.

This may be accomplished by burying the culvert invert. Energy disipators at the discharge end will usually be needed. Baffles in culverts may decrease velocities (also reduce pipe-full capacity) and may be useful in situations where culverts cannot be buried at near zero gradient.

The following charts show various relationships of flow, gradient, velocity and acres of watershed. The values are approximate and derived from Manning's Equation for bare round pipe. These charts are not for design purposes, but show that decreasing culvert gradients to meet fish passage requirements (ie velocities less than 1.5 fps) also significantly reduces culvert capacity, thereby requiring larger culverts if we are to design for both fish passage and design flows.

Example showing relationship of culvert size with grade, velocity and flow:

Size	Grade	Approximate low flow velocity, fps @ 0.5 cfs	Probable range of pipe-full cap; cfs
36"	5%	3.2 fps	85-95 cfs
	4%	3.0 fps	75-85 cfs
	3%	2.7 fps	65-75 cfs
	2%	2.4 fps	50-60 cfs
	1%	1.8 fps	35-45 cfs
fish passage requirement*	0.5%	1.5 fps	25-35 cfs
	0.25%	1.1 fps	15-25 cfs

Even though this table was derived using Manning's Equation, it clearly shows the inverse relationship between culvert grade and culvert capacity.

Culvert Size	Approx. Grade to Achieve 1.5 fps and corresponding cfs	Probable Range of Pipe-full cap; cfs	Approx. WS Area, 50 yr Event
18"	3.0%, 0.6 cfs	8-15 cfs	26 ac.
24"	1.5%, .13 cfs	12-20 cfs	34 ac.
36"	0.6%, .50 cfs	25-35 cfs	64 ac.
48"	0.4%, 1.0 cfs	50-60 cfs	115 ac.
60"	0.2%, 2.0 cfs	60-80 cfs	145 ac.
72"	0.15%, 3.0 cfs	82-110 cfs	200 ac.
84"	0.12%, 5.0 cfs	110-160 cfs	280 ac.
96"	0.10%, 6.0 cfs	140-190 cfs	363 ac.

* Approximate watershed area contibuting enough water to fill the pipe to a $\frac{HW}{D}$ ratio = 1.0 for a 50 yr. event.

Indications are that pipes laid on grades specified above will meet fish passage requirements 60 percent of the time on an annual basis.

You probably will not solve all your fish passage problems with the use of culverts on initial installation. A monitoring program will be necessary to assure passage of the Juvenile Fingerlings. Adjustments can be made and or designs perfected.

TABLE 1. DRAINAGE STRUCTURES TO BE USED FOR FISH PASSAGE

Type of Drainage Structure to be used to pass fish	Temporary Roads	Permanent Roads to be put in road storage	Permanent Roads not to be put in storage
Permanent Bridges		(x)	(x), x
Portable Bridges		x	(x)
Native Log Stringer Bridges	x	x	(x)
Native Rough Sawed Bridges	x	x	(x)
Log Culverts/Glu-Lam	x	x	0
Log Culverts/Temporary Deck	x	x	0
Open Bottom Arch			
1. Aluminum Integral Arch	0	x, 1)	x
2. Steel Arch	0	x, 1)	x
3. Half Round Pipe	0	x, 1)	x
Round/Pipe Arch Metal Culverts	*	*	*
Round/Pipe Arch Metal Culverts create natural bottom and construct plunge pools.	0	x, 1)	x
Baffled Pipe	0	x, 1)	x

x Acceptable for use (x) Combination permanent abutments and native log stringer or Bailey type bridge.
0 Not acceptable for use
* Optional if installed on a stream gradient of less than 2% and pipe size increased by the difference between that portion buried a minimum of 6" and the design discharge based on 25-year flood. Culvert should be installed on a grade of less than 1/2 percent.
1) Should be used only if structure is not going to be removed. (Must also be maintained).

Permanent Bridge - This type of structure is recommended for fish passage. Permanent bridges are recommended for use on permanent roads not to be put in road storage. A combination of permanent abutments and native log stringers or log abutments with removable superstructures can be used on roads that are put in road storage. The combination type bridge referred to is the Bailey and/or Acrow or the Hamilton Bridge.

Portable Bridge - Recommended for fish passage. The Bailey and/or Acrow and Hamilton portable bridge can be used with log abutments on temporary roads or roads to be put in storage. The bridges can be removed and reused readily. The portable bridge is usually installed on log abutments, but can also be a very satisfactory permanent installation when permanent abutments are used. See figures one and two.

Native Log Stringer and/or Rough Sawed Timber Bridges and Log Culverts - For initial construction costs and from a fisheries standpoint this is the most desirable drainage structure to pass fish. They maintain the natural stream bed and gradient. These structures should not cause any serious problem if properly constructed. The important thing with temporary structures is to see that they are promptly and correctly removed after log hauling. See Region ten's Design Guide for Native Log Stringer Bridges. For rough sawed timber bridges and log culverts see figures four and five.

Open Bottom Arch - This is the most desirable type culvert to pass fish and retain the natural stream bottom. We should try to keep the same bottom width as the natural channel. The important fact to remember is not to increase the stream gradient or stream velocities by restricting the natural channel width. The open bottom arch structure can be designed as: (1) aluminum integral arch, (2) steel arch, and (3) steel halfround. All of these three structures can be designed with various footings. See figure six.

Round/Pipe Arch Metal Culverts - May be used for fish passage if installed on a stream gradient of less than 2% and pipe size increased by the difference between that portion buried a minimum of 6" and the design discharge for 25-year flood. See figure seven.

Round/Pipe Arch Metal Culverts with Modifications - This structure is the most common and causes the most trouble in passing fish when used at stream crossings. Using this type of structure in designated fish streams requires modification in most cases. The modification involves using an enlarged culvert (12" to 18" larger than needed to carry a 25-year storm flow) placed 6 to 12 inches below the normal stream bed. This design reduces the slope of the culvert by burying it partially in the stream bed, allowing sediment (bedload) to deposit in the culvert. Sediment deposition will decrease the water velocity in the pipe and provide an irregular texture creating dart and test areas for small fish. A plunge pool can be designated to reduce the energy developed by the water as it passes through the pipe at higher flows. By increasing the end area of the channel, the velocity of the water is decreased and the energy dissipates as the fast water runs into the slow moving water in the pool. The controls (gabion, piling, walls, etc.) should be designed or built with the same life expectancy as that of the culvert.

It should be noted that when using flat bottom arch culverts the burial tolerances can be critical. The culvert design, round or arch, is based on the following criteria:

(1) Outlet buried minimum of 6" below natural stream gradient.

(2) Grade of culvert to be 3% less than natural grade of stream. If natural stream grade is 4% the designed culvert grade would be 1%. This leads to a significant amount of burial at the inlet end.

(3) The design is such that the end area of the culvert at the inlet end, above original stream bottom elevation, is sufficient to pass design flow.

With these criteria in mind the majority of available end area is located in the bottom portion of a pipe arch culvert. Since burial eliminates this area for designed flow caution must be used in selection of the arch pipe compared to the round pipe. See figure seven.

Use of Baffled Pipe - Constructing a fish passage device through a culvert essentially opposes the entire basic idea of the culvert, which is to discharge water downstream at the highest possible rate in relation to minimum culvert size required. A fish passage structure attempts to produce pockets of low velocity throughout the culvert where fish can rest. Providing these desired low velocities and resting spots requires some form of energy dissipators. Normally baffles or small water barriers of one type or another are used.

The best available information on baffle design is still found in the Washington Department of Fisheries report (McKinley and Webb, 1956). The principles developed at that time are sufficiently sound to be used as present guidelines, pending results of further research. We recommend the use of these designs.

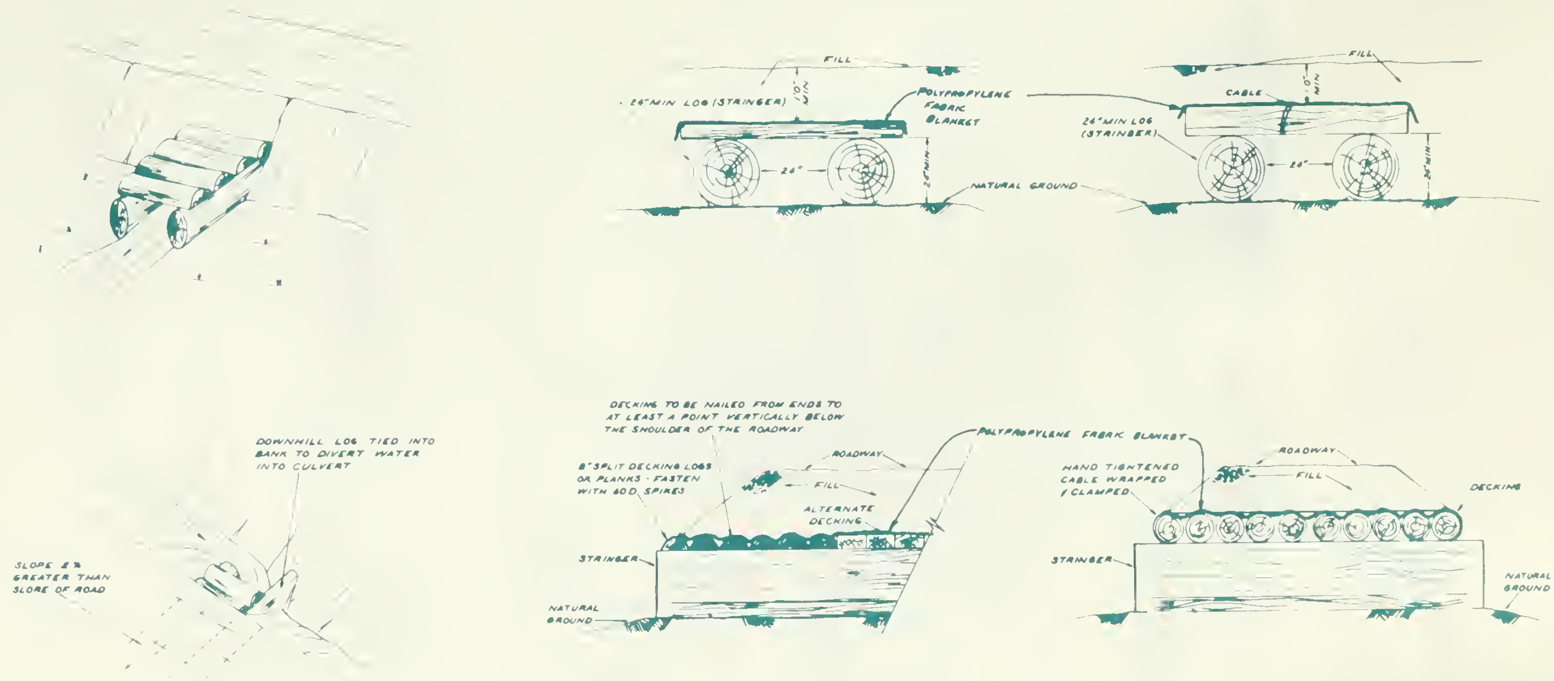
Figure 1

Figure 2

TYPICAL DETAILS
STANDARD LOG CULVERTS

NAILED DECK LOG CULVERT

ALTERNATE CABLE LASHED LOG CULVERT



TYPICAL DETAIL

NATIVE ROUGH SAWED TIMBER BRIDGE
16' WIDTH
12', 14', & 16' SPANS

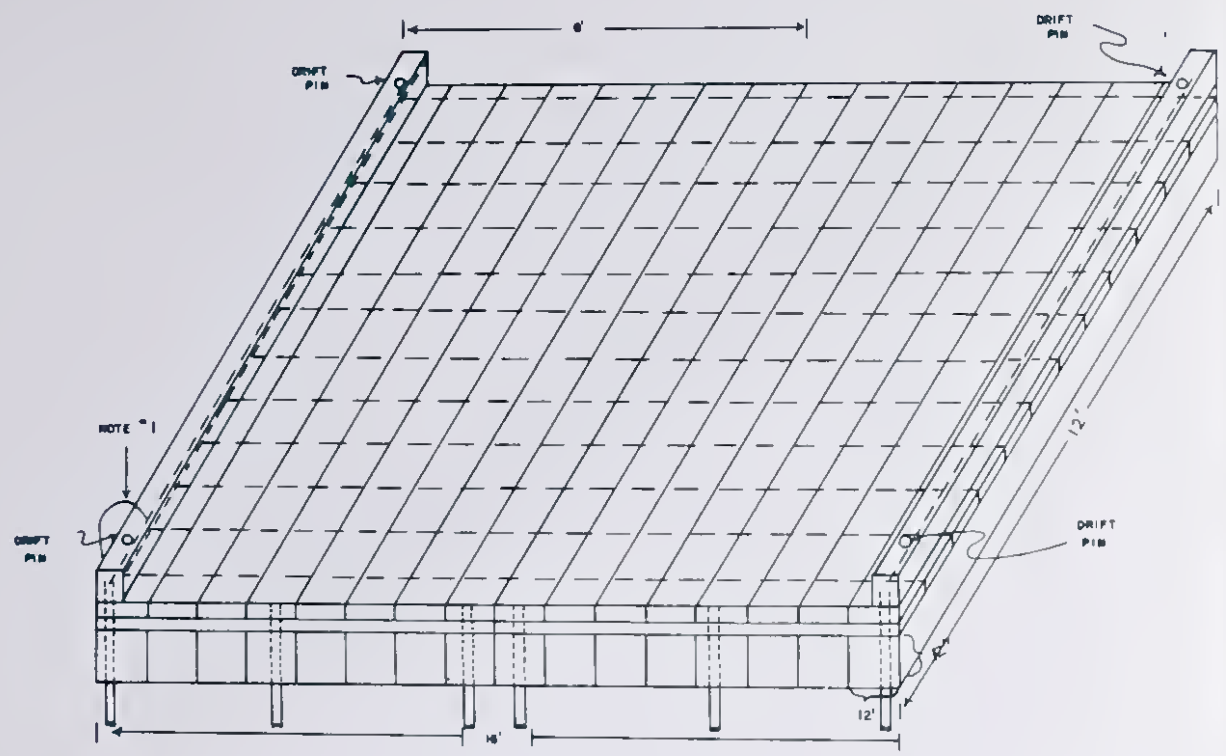
MATERIAL	SPAN 12'	14'	16'
Stringers	12" x 16" x 12' - 16ea	12" x 18" x 14' - 16ea	12" x 20" x 16' - 16ea
Bridge Deck	3" x 12" x 16' - 12ea 4" x 12" x 12' - 16ea	3" x 12" x 16' - 14ea 4" x 12" x 14' - 16ea	3" x 12" x 16' - 16ea 4" x 12" x 16' - 16ea
Wheel Guard	6" x 8" x 12' - 2ea	6" x 8" x14' - 2ea	6" x 8" x 16' - 2ea
Bolts	3/4" x 30" - 10ea	3/4" x 36" - 10ea	3/4 x 36" - 10ea
Washers	3/4" - 20ea	3/4" - 20ea	3/4" - 20ea
R. C. Nuts	3/4" - 20ea	3/4" - 20ea	3/4" - 20ea
Spikes	3/8" ø x 10" - 85#'s	3/8" ø x 10" - 97#'s	3/8" ø x 10" - 108#'s
Drift Pin	1 1/2" x 36" - 12ea	1 1/2" x 36" - 12ea	1 1/2" x 36" - 12ea
Cable	32 L.F.	32 L.F.	32 L.F.
Cable Clamps	8ea	8ea	8ea

CONSTRUCTION NOTES

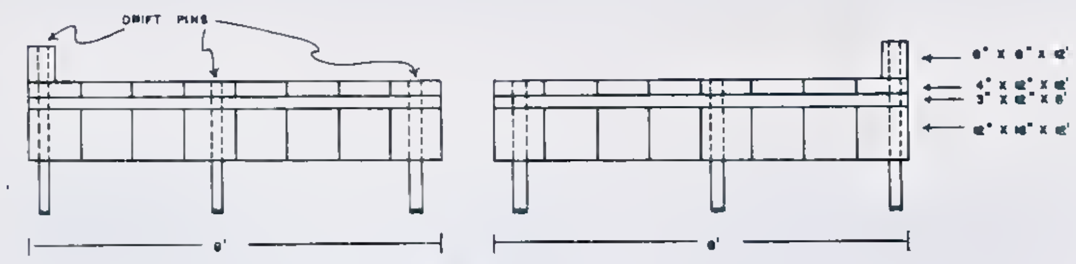
1. BRIDGE MAY BE ASSEMBLED ON SITE OR IS PREFABRICATED INTO HALF SECTIONS AND TRANSPORTED TO SITE (DRAWINGS INDICATE PREFABRICATION).

2. DRIFT PINS ARE USED TO ANCHOR BRIDGE TO SILL LOGS.

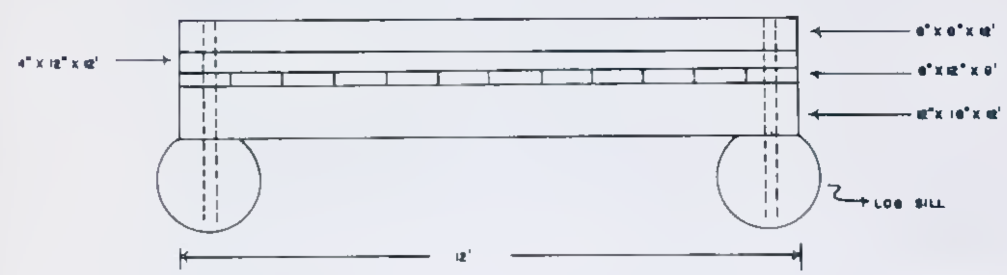
NOTE *1: 1" CABLE ON EACH 4 CORNERS TO LIFT WOODEN BRIDGE CULVERT



PLAN VIEW



FRONT VIEW



SIDE VIEW

FIGURE 3

FIGURE 4

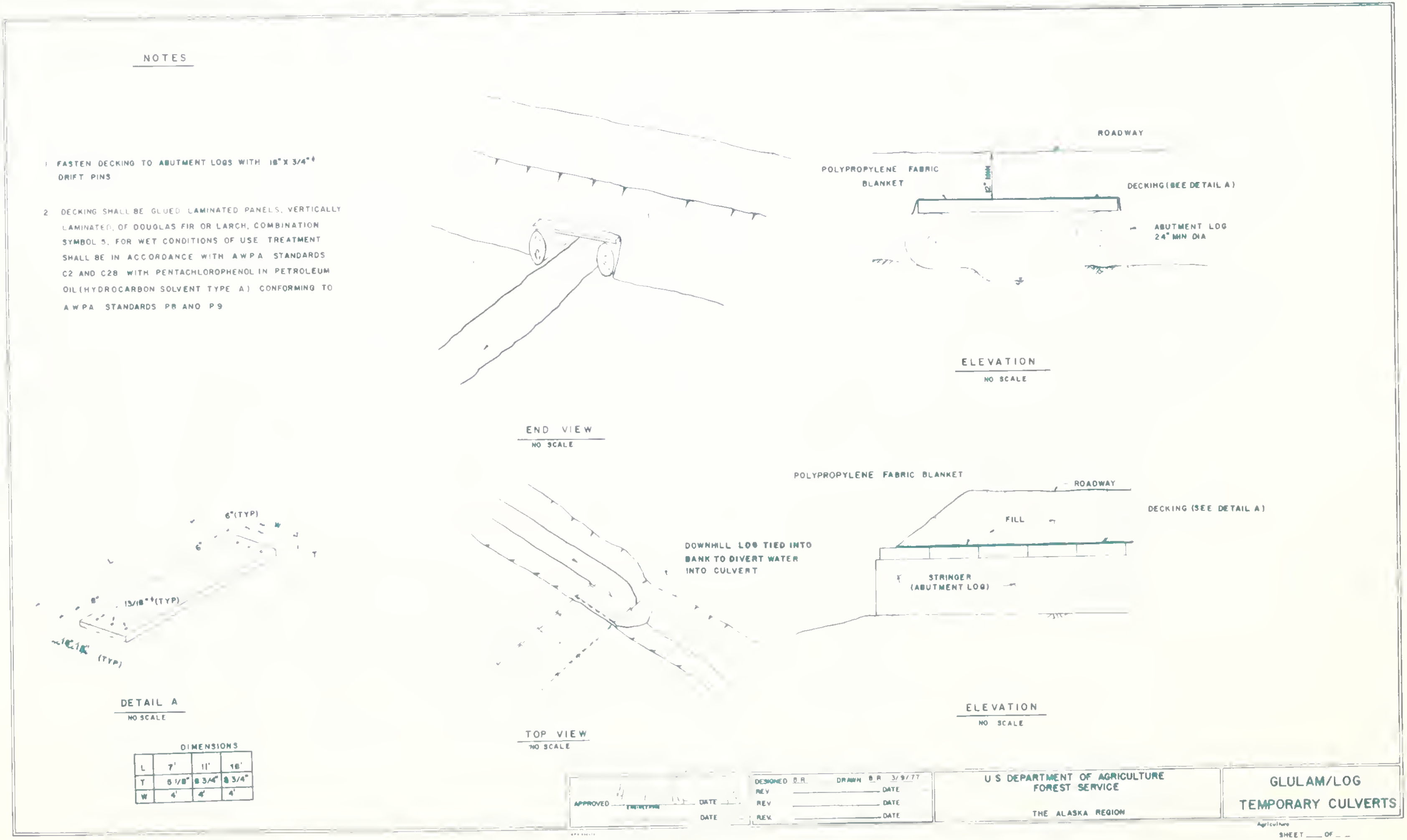
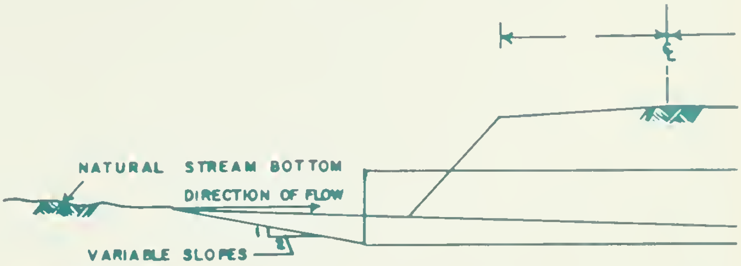


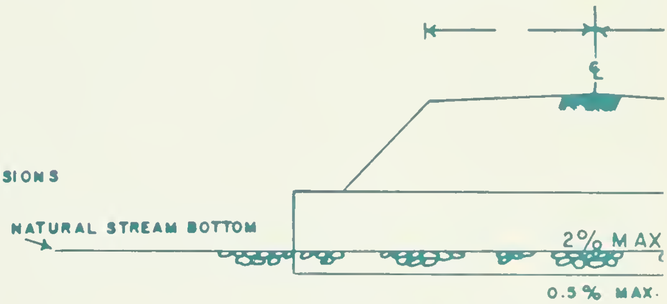
FIGURE 5

INSTALLATION O

IN FISH STRE



TYPICAL SECTION WIT



CULVERT TO BE INSTALLED (
NOT TO EXCEED 0.5%

STREAM GRADIENT NOT T(

TYPICAL SECTION WITH

NOTES

1 POOL SHALL BE CONSTRUCTED TO THE FOLLOWING DIMENSIONS

LENGTH = 3 X PIPE DIAMETER
WIDTH = 2 X PIPE DIAMETER
DEPTH = 1/2 X PIPE DIAMETER

2. BANKS OF POOL SHALL BE 2:1 OR AS STAKED.

3. LOOSE RIPRAP

FOR CULVERTS 48" IN DIAMETER AND SMALLER

MAXIMUM SIZE	40%	50%
200 LBS.	GREATER THAN 150 LBS.	GREATER THAN 75 LBS.

FOR CULVERTS 54" IN DIAMETER AND LARGER

MAXIMUM SIZE	40%	50%
400 LBS.	GREATER THAN 300 LBS.	GREATER THAN 75 LBS.

FIGURE 7

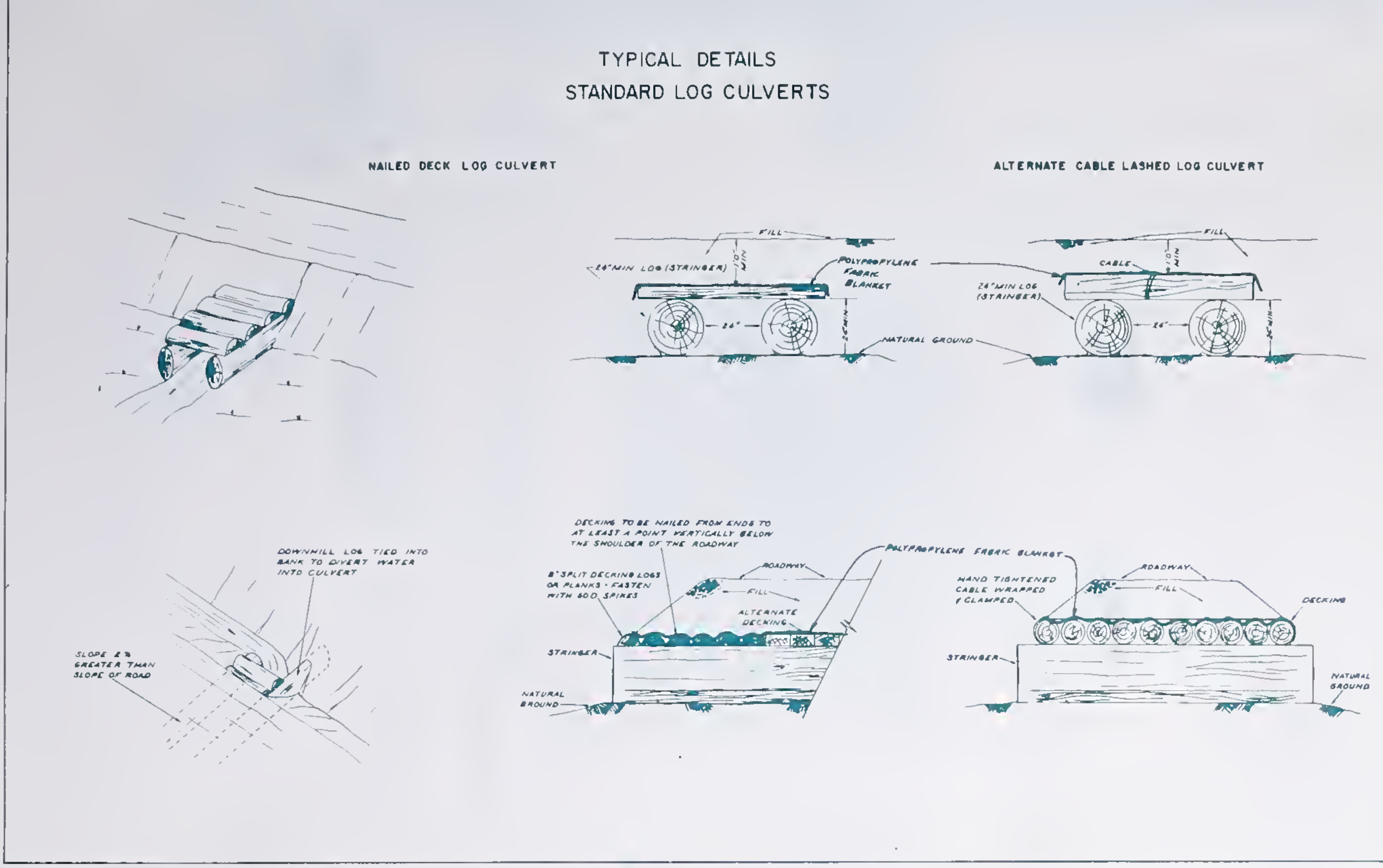


FIGURE 4

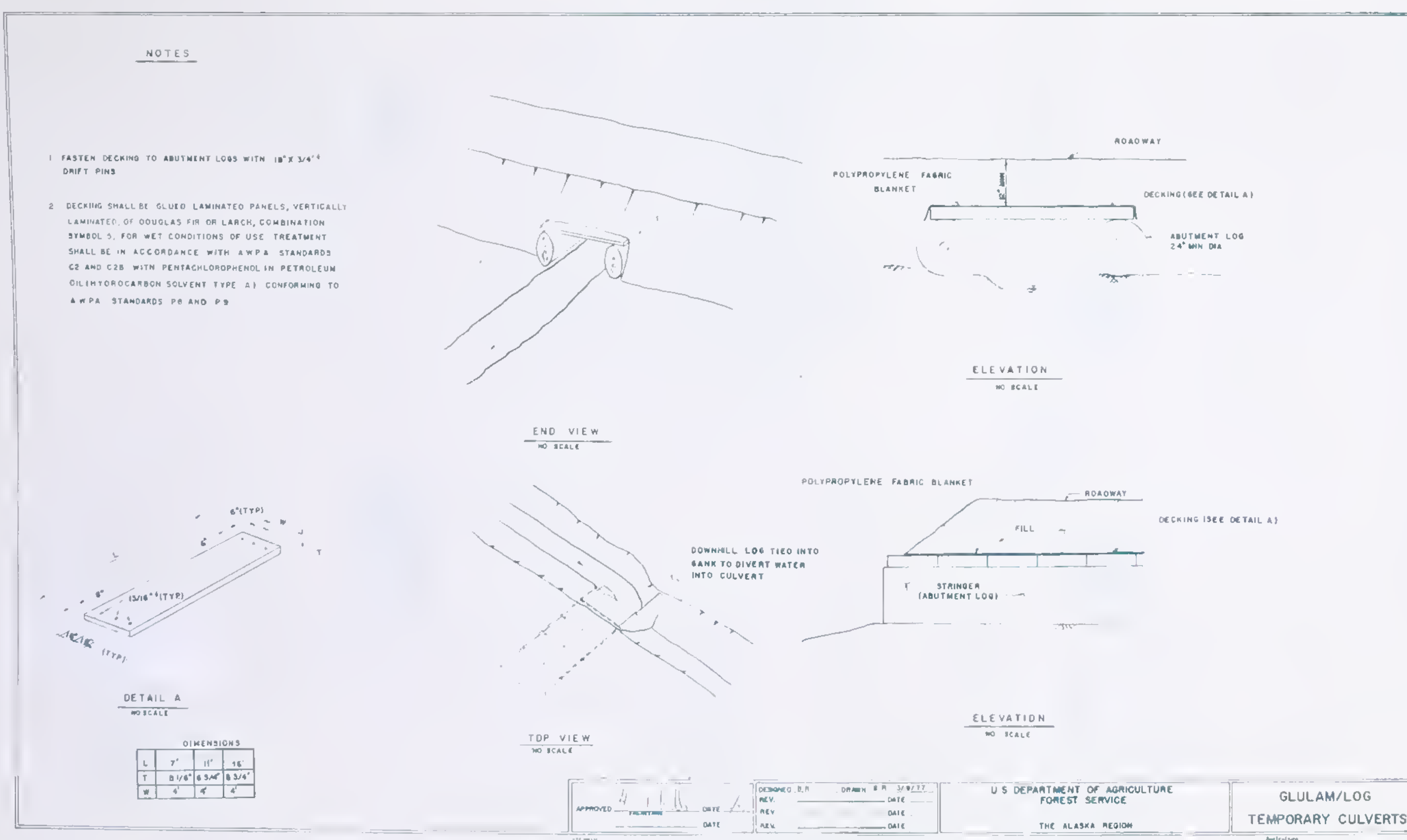
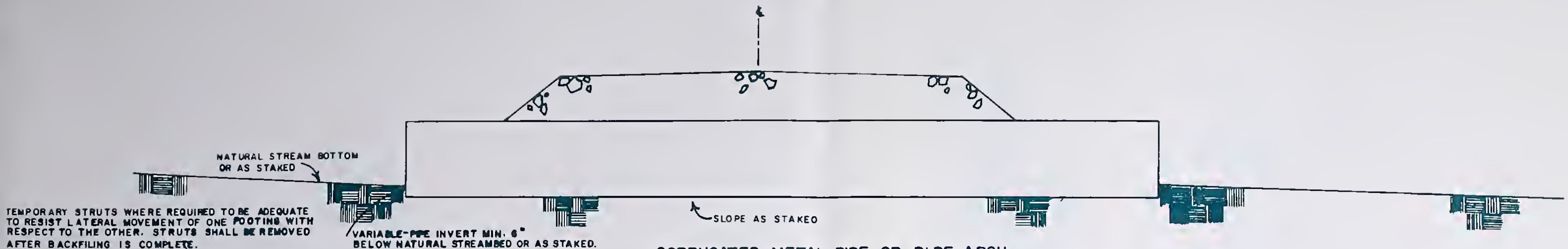
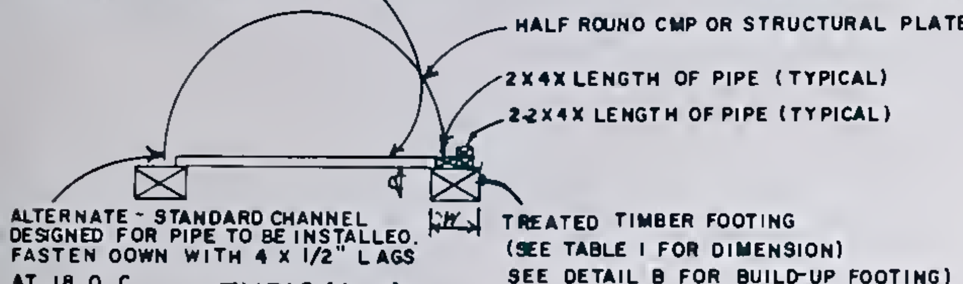


FIGURE 5



CORRUGATED METAL PIPE OR PIPE ARCH



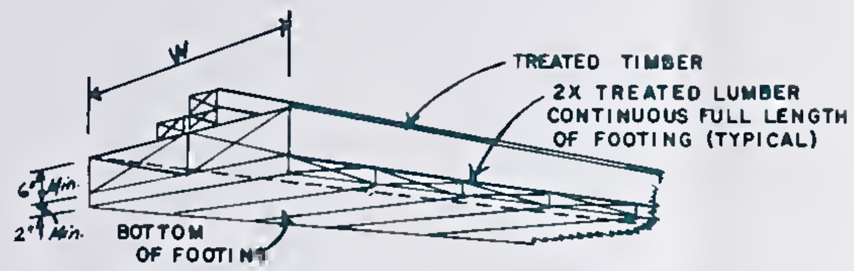
TYPICAL A

NO SCALE

TABLE I

TREATED TIMBER FOOTING SIZE

ARCH RADIUS	D	W
LESS THAN 36"	6"	12"
36"	8"	14"
42"	10"	18"
48"	12"	24"

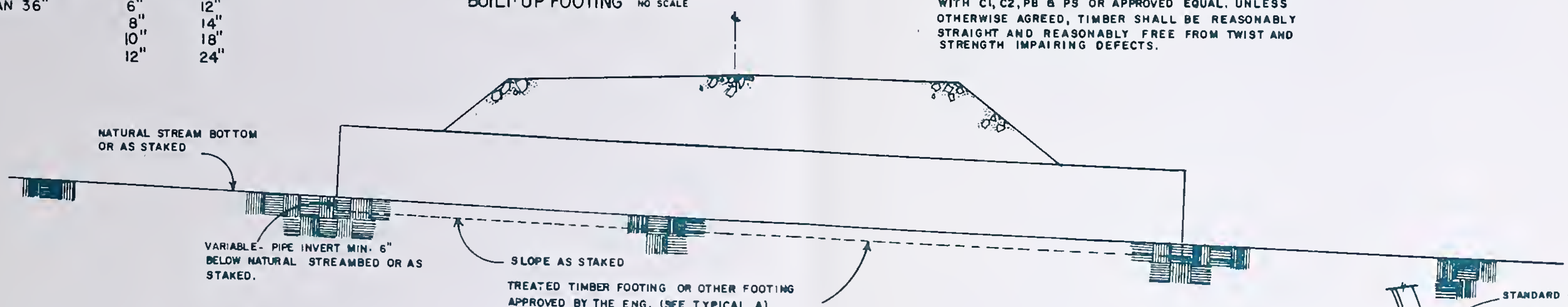


DETAIL B

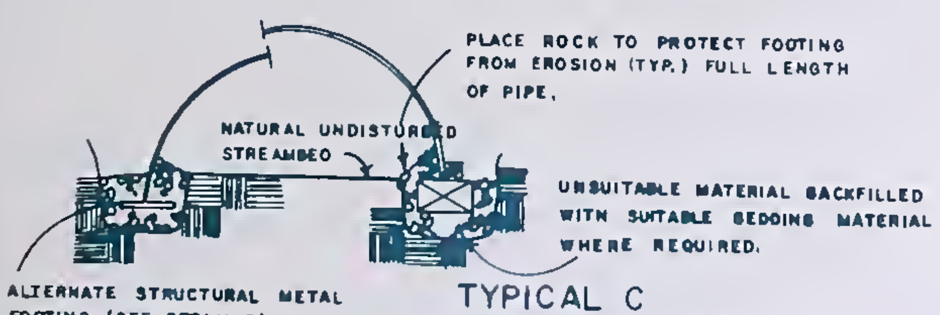
BUILT-UP FOOTING NO SCALE

GENERAL NOTES

1. COVER OVER THE TOP OF ALL STRUCTURES SHALL BE A MINIMUM OF 1.0 FT. UNLESS OTHERWISE INDICATED
2. STRUCTURES SUBJECT TO CONTACT WITH SALT WATER (WITHIN TIDAL ZONE) SHALL BE BEDDED AND BACK-FILLED WITH CLEAN ROCKY MATERIAL.
3. ALL STEEL AND FASTENERS SHALL BE GALVANIZED UNLESS OTHERWISE INDICATED. IF PIPE IS ALUMINUM, ADJACENT METAL PARTS SHALL BE ALUMINUM. FASTEN CHANNEL TO STRUCTURAL METAL FOOTING AND CULVERT WITH 6061 - T6 ALUMINUM OR A307 OR A325 STEEL.
4. ALL TIMBER SHALL BE TREATED WITH A 10% SOLUTION PENTACHLOROPHENOL IN HEAVY OIL, IN ACCORDANCE WITH C1, C2, P8 & P9 OR APPROVED EQUAL. UNLESS OTHERWISE AGREED, TIMBER SHALL BE REASONABLY STRAIGHT AND REASONABLY FREE FROM TWIST AND STRENGTH IMPAIRING DEFECTS.
5. 2 IN. LUMBER NAILED TO TIMBER FOR SPLICING, ETC. SHALL BE NAILED WITH 16D NAILS SPACED AT A MAX. OF 10 IN. WITH NO LESS THAN 4 NAILS TO EA. BOARD.
6. EXCEPT AS OTHERWISE INDICATED, THE STREAM CHANNEL SHALL BE MAINTAINED TO ITS NATURAL GRADIANT, ROUGHNESS, AND CONDITION.

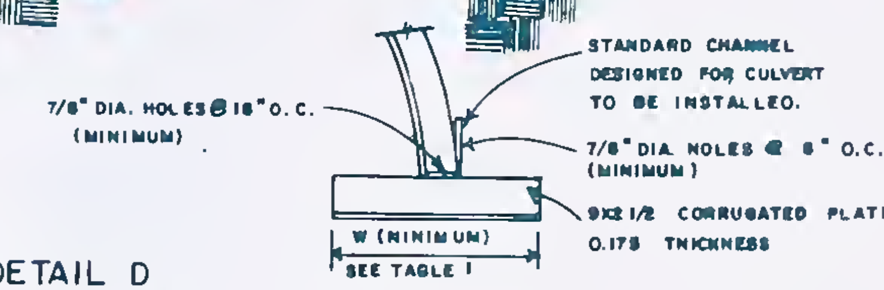


INTEGRAL ARCH



TYPICAL C

NO SCALE

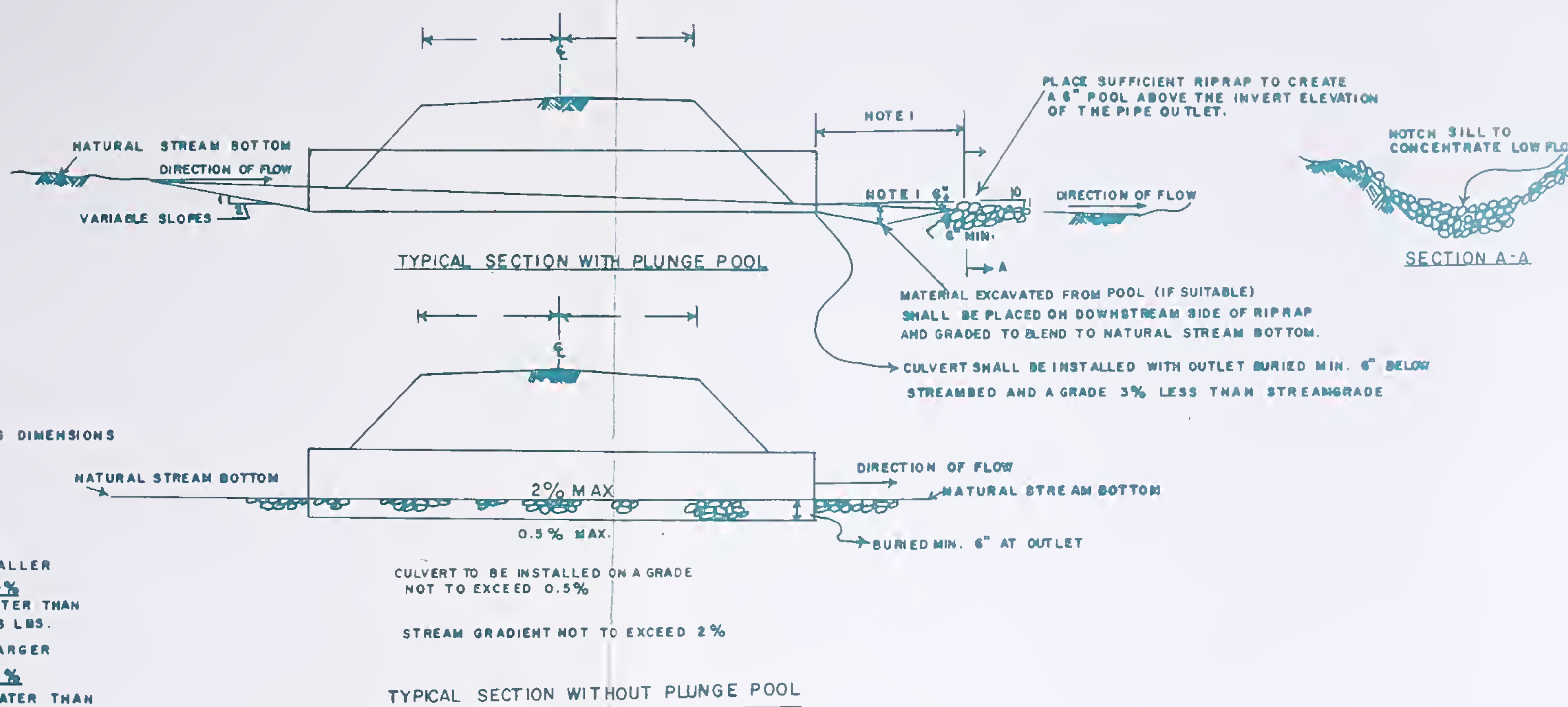


DETAIL D

ALTERNATE STRUCTURAL METAL FOOTING NO SCALE

FIGURE 6

INSTALLATION OF CULVERT IN FISH STREAM



NOTES

1. POOL SHALL BE CONSTRUCTED TO THE FOLLOWING DIMENSIONS

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   FOR CULVERTS 48" IN DIAMETER AND SMALLER

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200 LBS.	GREATER THAN 150 LBS.	GREATER THAN 75 LBS.

   FOR CULVERTS 54" IN DIAMETER AND LARGER

MAXIMUM SIZE	40%	50%
400 LBS.	GREATER THAN 300 LBS.	GREATER THAN 75 LBS.

FIGURE 7

APPENDIX: PART A

DRAINAGE STRUCTURE SURVEY HANDBOOK

INTRODUCTION

A. General

Procedures for the preparation of drainage structure site surveys differ from area to area and even between project engineers, but basic fundamentals are common throughout. Adherence to fundamentals is exemplified in the clearness and reliability of the finished product.

The site survey is needed to determine the most economical, aesthetic and environmentally suitable structure for the site. The data compiled must be accurate and complete, thereby aiding the design engineer in selecting the proper structure for the site. In addition, errors in the work may result in monetary claims by the contractor, with the loss of time by everyone involved.

B. Application of This Handbook

The procedure presented herein is intended basically as a guide. The Project Engineer and the various specialists involved are encouraged to use their own methods and ingenuity to achieve the accuracy and completeness required.

The requirements presented herein differ depending on the site, but the requirements cited are the minimum needed for proper design. Any additional information that the person in the field can supply is potentially useful in designing an adequate drainage structure.

FISH STREAMS

The presence of salmonoids in a stream is significant in the design, construction and maintenance of a drainage structure.

A fish stream not identified as such may lead to improper design and installation of a road crossing. This could result in a blockage to fish migrations and excessive sedimentation. Corrective action, nearly always required, is far more expensive than proper installation and imposes additional stress on the stream system.

The primary criteria for identification of a fish stream is the presence or absence of fish utilizing the area above the road crossing. In the absence of a written report by a fisheries biologist, a stream should be designed to maintain a fish population.

SMALL CULVERTS (LESS THAN 35 SQUARE FEET)

A. General

Small culverts may be log culverts, C.M.P., or C.M.P.A. The particular type used depends on the category of road, the presence or absence of fish, the stream gradient and other factors.

B. Survey Data

The following data are required for design:

1. Road Station at stream centerline.
2. Width and depth of stream.
3. Velocity of stream and stage of water.
4. For an approximate distance of 100 feet upstream and downstream and to an accuracy of 1:300, the following information is required:
    a. Elevations of stream bottom
    b. Elevations of stream banks
    c. Elevations of edge of water
    d. Elevations of high water marks
5. An estimate of Manning's n for the reach for both high and low flows.
6. Sketch showing all pertinent features describing the drainage basin, general slope, length of cover, aspect, etc. Sufficient for hydrologic analysis (design hydrograph).
7. Fish Stream Survey Report, if applicable.

All elevations are to be based on the same datum as the road survey, when possible, or an assumed datum. Whether a stream is to have a culvert or be bridged may not be readily apparent to the survey crew or Project Engineer. When in doubt, the most efficient approach is to assume a stream will be bridged. That assumption requires more data from the survey crew but gives the designer a clearer picture of the area.

TEMPORARY BRIDGES

A. General

Except in rare instances, most logging road bridges are classified as "temporary". Bridges in this category will usually have log cribbing for abutments. The superstructures may be log stringers or "portable" steel bridges.

B. Survey Data

The following data are required for temporary bridges. All data are required to an accuracy of 1:300.

1. All the data required for small culverts.
2. For an approximate distance of 75' on both sides of the stream centerline provide enough data to construct a contour map of the area with a contour interval of 2 feet.
3. Provide benchmark (spike in tree base) for an elevation reference out of range of future construction activities. This elevation must be tied to the road survey.
4. Provide a reasonably accurate sketch showing major topographic features, road, grade breaks, hub, etc.
5. Fish Stream Survey Report, if applicable.

C. Supplementary Data

The Project Engineer may choose to enlist the services of the area hydrologist and biologist or other specialists in the compilation of the following material:

1. Bridge Site Data Sheet, Form R10-7700-30.
2. Fish Stream Survey Report.
3. Hydrologic and Hydraulic Analysis. This report should contain the following:
    a. Estimated discharge volumes for design flood frequency discharges.
    b. Anticipated scour and recommended means to control scour.

c. Recommended skew of substructure.

d. Recommended freeboard clearance.

e. Recommended channel improvements, such as riprap, dikes or relief structures.

4. The following information is optional:

a. Photographs.

b. Terrain features which favor or preclude certain bridge types. For example, Acrow (Bailey) type bridges need space for a launching area.

c. Roadway design information.

PERMANENT STRUCTURES

A. General

Permanent structures include major culverts, which are defined as all culverts greater than 84 inches diameter (35 square feet or greater waterway opening). A steel, concrete or glued-laminated timber bridge is the usual design solution for a permanent bridge. In some cases, the permanent structure will be replacing a temporary bridge. The Project Engineer may be able to get preliminary site information from data used to build the original temporary structure.

B. Site Research

By doing adequate preliminary office research, only one trip to the site may be necessary. A minimum check list to consult might be as follows:

1. "As built" plans.

2. Existing bench marks.

3. Existing survey monuments.

4. Plans of existing bridge, if applicable.

5. USGS map

6. Maintenance records, if applicable.

C. Survey Data

The accuracy required is an allowable error of closure of 1:2000 horizontally and an allowable vertical error of 0.5 feet per mile. The following data are required:

1. Complete Bridge Site Data Sheet, From R-10-7700-30.

2. Sufficient data to construct a contour map of the area with a coverage of 100 to 150 feet beyond the extreme limits of the structure. The contour interval should be 1 foot.

3. Centerline profile of the road for 100 to 150 feet beyond the structure.

4. Stream X-Sec. parallel to road, 15'Rt. and Left. Additional cross-sections perpendicular to stream channel are necessary to determine slope of channel, between 500 ft upstream and 500 ft downstream.

5. Bench marks, spike in base of tree, established out of range of future construction activities for re-establishing vertical control. Monuments (hubs and tacks) should be established for horizontal control.

6. Fish Stream Survey Report, if applicable.

D. Supplementary Data

1. Fish Stream Survey Report, same as for temporary bridges.

2. Hydrologic and Hydraulic Report, which should include the following:

   a. Estimated discharge volumes for design flood frequency intervals.

   b. Anticipated scour and recommended means to control scour.

   c. Recommended skew of substructure.

   d. Backwater and analysis for proposed bridge sizes and flood frequency intervals.

   e. Recommended freeboard clearance.

   f. Recommended channel improvements such as dikes, riprap, etc.

3. Foundation Investigation Report.

4. Photographs (10 to 15) of site.

SUMMARY

The preceding has been an attempt to enumerate the main points to consider in the data collecting process for the design of drainage structures.

The following material should be provided to the designer of the drainage structure.

A. Small Culverts

1. Plan and profile of stream and drainage structures. This is generally part of a set of plans for road construction.

2. Fish Stream Survey Report, if applicable.

B. Temporary Bridges

1. Plan and profile of stream and drainage structures.

2. Topographic map, 2 foot contour, scale of 1 inch equals 10 feet. Show all pertinent features.

3. Fish Stream Survey Report, if applicable.

4. Bridge Site Data Sheet.

5. Hydrologic and Hydraulic Report.

C. Permanent Structures

1. Topographic map, 1 foot contour, scale of 1 inch equals 10 feet. Do not use Federal-Aid Sheet.

2. Fish Stream Survey Report, if applicable.

3. Foundation Report with drill logs.

4. Plan and profile of road centerline; scale of 1 inch equals 10 feet.

5. Cross-sections of stream; scale of 1 inch equals 10 feet.

6. Hydrologic and Hydraulic Report.

7. Photographs of site (10 to 15).

8. Roadway Design Information with recommended alignment, finished grade and typical road cross-section.

9. Vicinity map.

10. Bridge Site Data Sheet.

REFERENCES

The following references may help clarify or supplement the material presented herein:

1. FSH 7709.11, chapter 82.

2. FSH 7709.11, chapter 71.1 and 71.3.

3. FSM 7717, Drainage Engineering.

4. FSM 7723, I.D. No. 3, Structures.

5. R-10, "Foundation Investigation and Design" by W.A. Vischer.

FISH STREAM SURVEY REPORT

Forest ______________________________ Stream Name (ID) ___________

Road Trail Name ________ No. _______ Mile Post ________

Fish Present Yes _________ No ________

Above road crossing _________ ________

Below road crossing _________ ________

Species Present ____

____

If fish are not present, is future habitat development possible?

Natural Barriers to Migration: Yes __________ No __________

If "yes" describe:

Can site be improved by removal of natural barriers?

Would you recommend an alternate stream crossing site in the vicinity:
Describe:

If resident fish population present, estimate number in the vicinity:
Show Sketch:

Methods used to determine presence of fish:

Date Gear Type Length of Time Fished

Type of fish habitat:

Rearing Area: ___________ sq yd Describe __________________________

Spawing Area: ___________ sq yd Describe __________________________

Data obtained from other sources:

No _____________ Yes _____________

Remarks: List construction activity restrictions at this site such as time of year, equipment in stream, etc.

Reporting Biologist (Name)

Signature ________________________________

Date _____________

USDA - Forest Service

R10-7700-30
April 1975

Region 10

BRIDGE SITE DATA SHEET

(see FSH 7709.11, ch. 82.4)
This data sheet is to accompany a contour map of the site and a profile along centerline (and 15 ft. Lt. & Rt.) of roadway.

Forest ____________________ Stream Name ____________________
☐ Road ☐ Trail Name ____________________ No. ________ Mile Post ______
Bench Mark (Location & Descrip.) ____________________
____________________ B.M. Elev. ____________________
Location - Sec ____ T ____ R ______ Mer. ____________ Or Lat. & Long.
or State Plane Coord. ____________________

DRAINAGE AND FLOW DATA

Drainage Area ________ Gradient of Stream _____% Slope of watershed ___%
Type of Soil ____________________
Type of Cover ____________________
Will (will not) be logged off _____%. Max. rainfall or runoff __________
Nearest gaging station ____________________
Cause and season of floods ____________________
Banks or bed show scour? ____________________
Drift or ice (amount & character) ____________________
Has stream glaciered at this site? __________ Amount __________ Date ______
Stream Type (Old, meandering, youthful, straight, etc.) __________
____
____
Average particle "size" of bedload ____________________

	Date	500 ' Upstream	At Site	500' Downstream
Extreme Ice Elev.				
Extreme H.W. Elev.				
Ordinary H.W. Elev.				
Normal Water Elev.				
Low Water Elev.				
Streambed Elev.				

Estimated Velocity - Normal Water ______ H.W. ______ Ext. H.W. ______
Method of Determination ____________________
Source of Information ____________________

FOUNDATION DATA

Foundation Conditions (Describe investigation made and attach Foundation Report/Analysis) ____________________
____
____
____
____

EXISTING STRUCTURE

Type, No. of spans, and lengths ____________________
____________________ Plan No. ____________
Type of Substructure ____________ Condition ____________
____
Waterway opening ____________ Adequate? ____________
Structure affected by debris, ice, scour? ____________
____________ Does structure constrict channel? ____________
Does backwater condition exist? (Describe) ____________
Has structure settled? ________ Depth of ftgs or piling ________

PROPOSED STRUCTURE

Type ____________________
Length ________ Width ________ Loading ________
Length of channel span ________ Vert. clearance above H.W. ________
Type of substructure ________ Approx. length piling ________
Channel change recommendations (show on site plan) ________
____
Are dikes or bank protection recommended to control flow? (show on site plan)
____

MISCELLANEOUS DATA

Sources: Conc. aggregate, Ready-mix conc. (if available), Riprap, Embankment borrow ____________________
____
____
____
____
Location & recommended length of detour ____________
____
____
Status of Right-of-Way ____________________
Is site in power withdrawal area? ________ Clearance obtained? ________
Use by boats (amount & type) ____________
List utility lines and other facilities at site and show ownership ______
____
____
____
Remarks: ____________________
____
____
____
____
____

Surveyed by ____________ Date ____________

Reviewed by ____________ Date ____________

Approved by ____________ Date ____________
(Forest Engineer)

GSA524

APPENDIX--PART B

TRANSPORTATION ENGINEERING HANDBOOK

CHAPTER 70 - DRAINAGE ENGINEERING

71.22 - Culverts

5. When considering the use of culverts where fish passage is required, the designer should consider the pipe flow velocities not only at full flow capacity but at partial flow capacities also. This will enable one to more fully examine the suitability of the culvert for all required sizes of fish at various critical periods of the year. In order to do this, the designer must obtain the necessary input from the Fisheries and Hydrology Disciplines.

71.33 - Culvert Location

12. In cases where it is necessary to place the structure on very weak, compressible subgrades such as muskeg (peat), appropriate settlement and bearing capacity considerations should be made to assure the proper final grade.

13. A fish stream is defined as any water flow that is accessible to fish and capable of supporting aquatic lift. This includes, but is not limited to, all Alaska Department of Fish and Game designated streams and all their tributaries up to impassable natural barriers. Freshwater systems above blockages may also support resident fish stocks. Evaluations and recommendations will be made by a fisheries biologist during route locations to determine the presence of fish stocks.

The placement of culverts, with regard to ensuring continued fish passage in stieams should assure the following conditions:

a. The fish streams will be managed to ensure maintenance of stream velocities which will permit upstream movement by juvenile salmonids in all streams and tributaries not blocked by impassable barriers, as determined by a fisheries biologist. Stream velocities at low flows are most critical from a fish passage standpoint. They often coincide with significant periods of upstream movement and in-migration. A graph of juvenile salmonid swimming capabilities (Fig. 1) has been compiled to serve as a guideline for proper velocities through culverts at low flow levels. The magnitude of the speed of a 50 mm. (1.97 inch) salmonid which is approximaιely 0.5 ft/sec. to 1.0 ft/sec.

b. Culvert outlet conditions must allow unimpeded access by juvenile salmonids. The jumping ability of juveniles is assumed to be zero, and there must therefore be no plunge of waterfall at the culvert outlet. Controls will be installed, if necessary, to maintain the downstream water levels above the base of all pipe culverts. Location of culverts is very important. Select a road crossing where there is no sudden increase in water velocity above, below or at the crossing location.

71.44 - Construction

2. Camber. If excessive settlements are anticipated such as in muskeg terrain, it may be unfeasible to consider camber initially and preloading rechniques may be required prior to final culvert installation. A typical example of pre-loading would be where fills across small muskeg drainages are first placed without the cross drain to allow for settlements due to the fill load. After one to two weeks most of the potential settlement will have occured. At this point the fill can then be excavated, the drainage structure installed and the fill replaced. If the preloading time has been sufficient, further settlement should be negligible in most cases.

3. Backfilling. If course-grained granular materials such as pitrun rock is used for backfill, the above control procedures are often unfeasible, hence other appropriate methods must be used. One method would be using test procedures more compatible with granular material or merely using a small control strip or section with nuclear density correlations. Often times if one has sufficient knowledge of the material, specified equipment passes will suffice.

74 - BIBLIOGRAPHY

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Flow duration tables through 1974; A U.S. Geological Survey computer printout of high and low flows, based on mean daily flows both based on the log-Pearson Type III distribution analysis. Part 1 includes and plots of 7-day and 30-day low flow recurrence interval. Part 2 includes tables and plots of flood frequency. Part 2 includes tables and plots of flood frequency. Part 1 is updated by the USGS at periodic intervals, while Part 2 can be updated annually for active stations. Copies can be requested through Regional Office, Resource Management Unit.

JUVENILE SALMONID
SWIMMING CAPABILITY

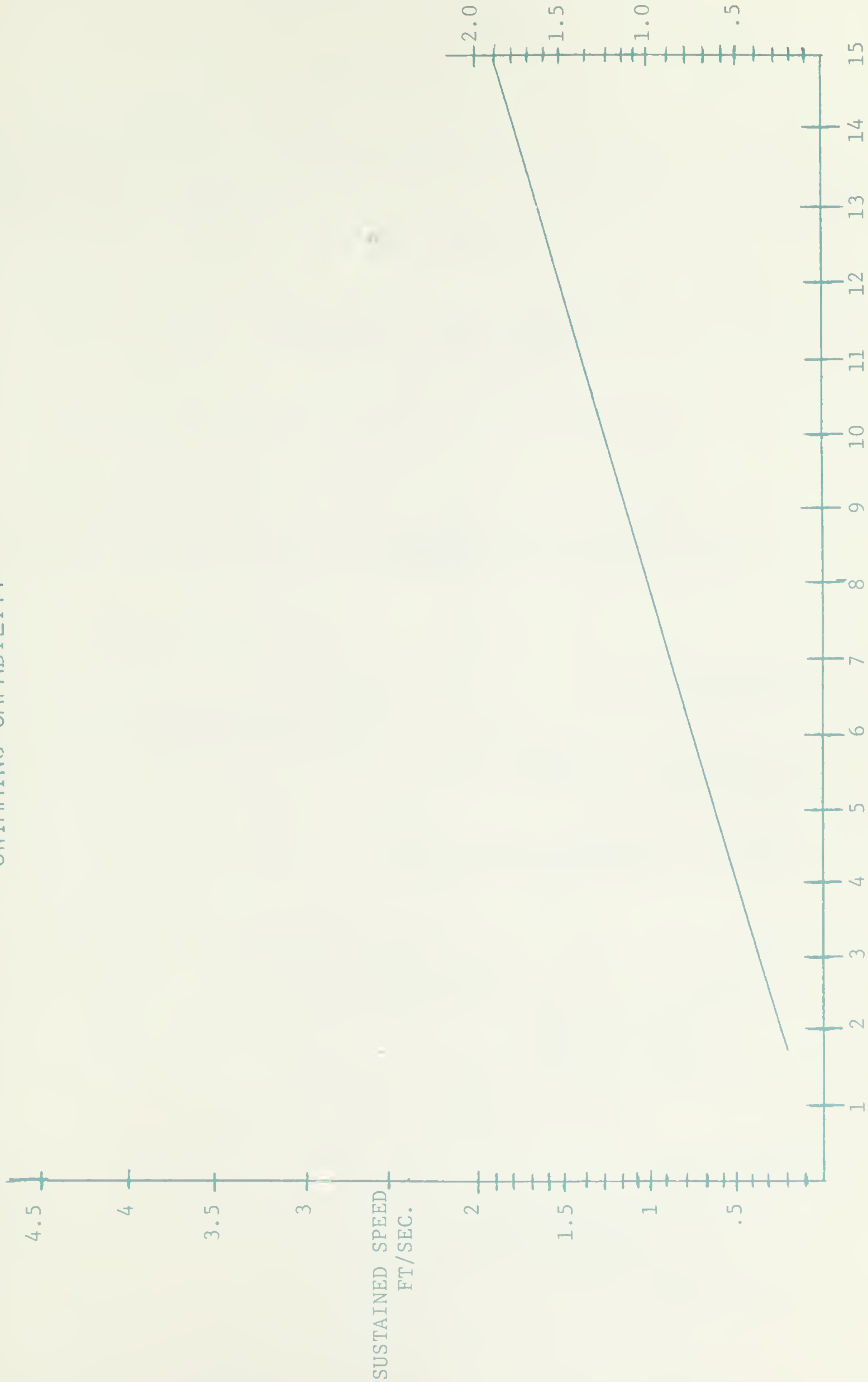


Figure No. 1

APPENDIX--Part C

DESIGN GUIDE FOR DETERMINING THE DESIGN FLOW FOR HYDRAULIC STRUCTURES IN SOUTHEAST ALASKA

U.S. Department of Agriculture
Forest Service, Region 10
(Revised March 1977)

CONTENTS

INTRODUCTION

This guide outlines the recommended methods for determination of the design flow for U.S. Forest Service hydraulic structures in Region 10, Southeast Alaska. It is based on the methods and data presented in the following publications:

1. "A Uniform Technique for Determining Flood Flow Frequencies", Water Resource Council, Bulletin No. 15, December 1967.

2. "Flood Frequency in Alaska", U.S.G.S. Water Resources Division, Open File Report, 1970.

3. "Magnitude and Frequency of Floods in Alaska, South of the Yukon River", U.S.G.S. Circular No. 493, 1964.

4. "Hydraulics Manual", State of Alaska Department of Highways.

As additional stream flow data is continually being gathered, this guide should be updated periodically to reflect the most current developments.

Updated for Ketchikan Area 1/77
by Louis R. Bartos, pages 3-10 and 17-26

Updated for Stikine and Chatham Areas
Have not been completed as of date of publication.

PROCEDURE

The procedure for design flow determination consists of:

1. Determining the Design Frequency for the structure.

2. Determining the Design Flow for the structure using the Design Frequency.

DESIGN FREQUENCY

Economical design of drainage structure requires a knowledge of the frequency and magnitude of peak rates of runoff. Since it is normally not economical to design a structure for the maximum peak runoff, it is necessary to take some risk that a flood greater than the design flood will occur during the life of the structure. Figure 1 from chapter 70 of the Transportation Engineering Handbook is included and relates the risk involved for various design periods and return intervals.

The choice of a design flood must also consider all the economics involved, including not only the initial cost of the structure but the amount of damage incurred and maintenance necessitated if the design flood is exceeded. Such an analysis should be required on any major structure, however the following guidelines as outlined in section 7717 of FS Manual 7700, can be used as general criteria.

Structures	Design Frequency
Minor	10 yr. minimum without head 25 yr. fish passage 50 yr. with allowable headwater
Major Culverts (End area 35 ft. and Minor Bridges (length 30")	25 yr. (50 yr. check)
Major Bridges	50 yr. (100 yr. check)

For a more detailed discussion refer to the FS Manual.

3-4

GRAPHIC METHOD FOR DETERMINING DESIGN FLOWS FOR CULVERTS AND BRIDGES IN SOUTHEAST ALASKA

By
Louis R. Bartos
Hydrologist

USDA Forest Service, Region 10
January 1977

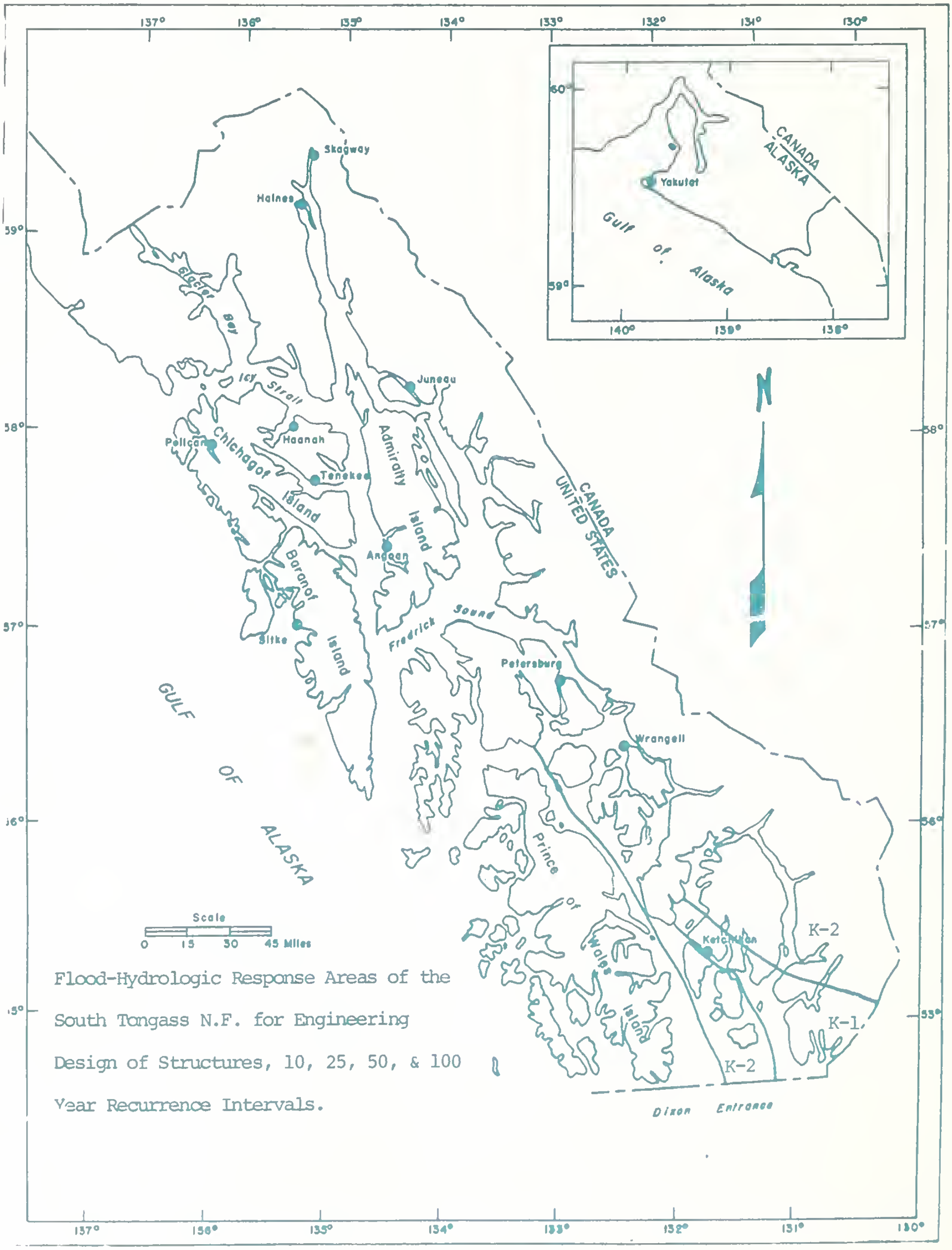
The following guide, for the present time, is the recommended method for determining the design flow to establish end areas for culverts and small bridges in southeast Alaska. It should be noted that this paper should be used in conjunction with the FHWA, Hydraulic Engineering Circular No. 5 for determining culvert size.

<u>How to use this paper</u>:

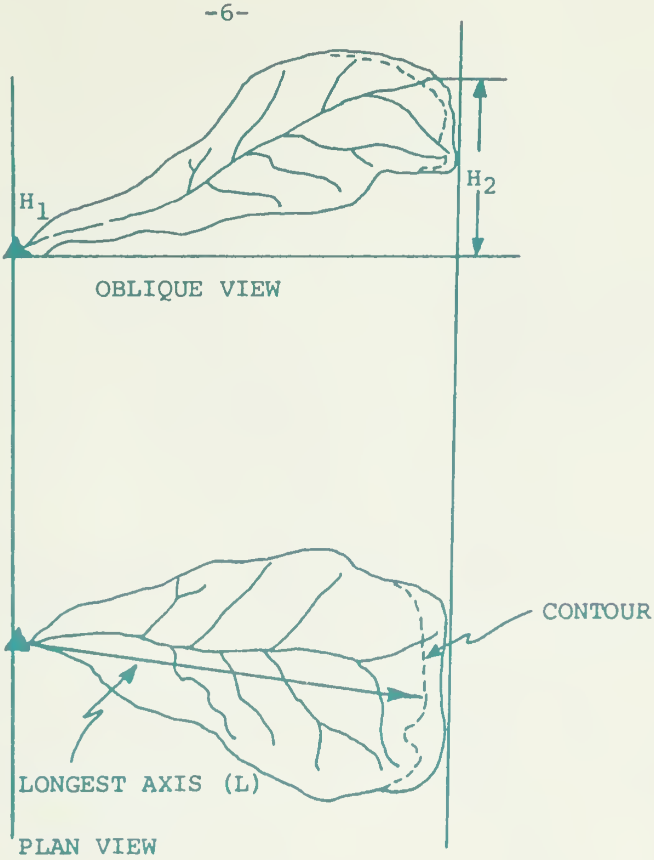
Data needed:

(a) Drainage area in square miles
(b) Average drainage slope (can be obtained from a topographical map)

The drainage area and the drainage relief are determined from the point of the project. The drainage relief is the difference in elevations between the project and the uppermost drainage point of the longest axis. The longest axis (L) is that line within the drainage, without crossing the topographical boundary, from the site of interest to the longest contour (Figure 1). The drainage relief factor is the square root of the drainage relief. Elevation affects the flow of water by both flow energy and amount of precipitation because of orographic effects.



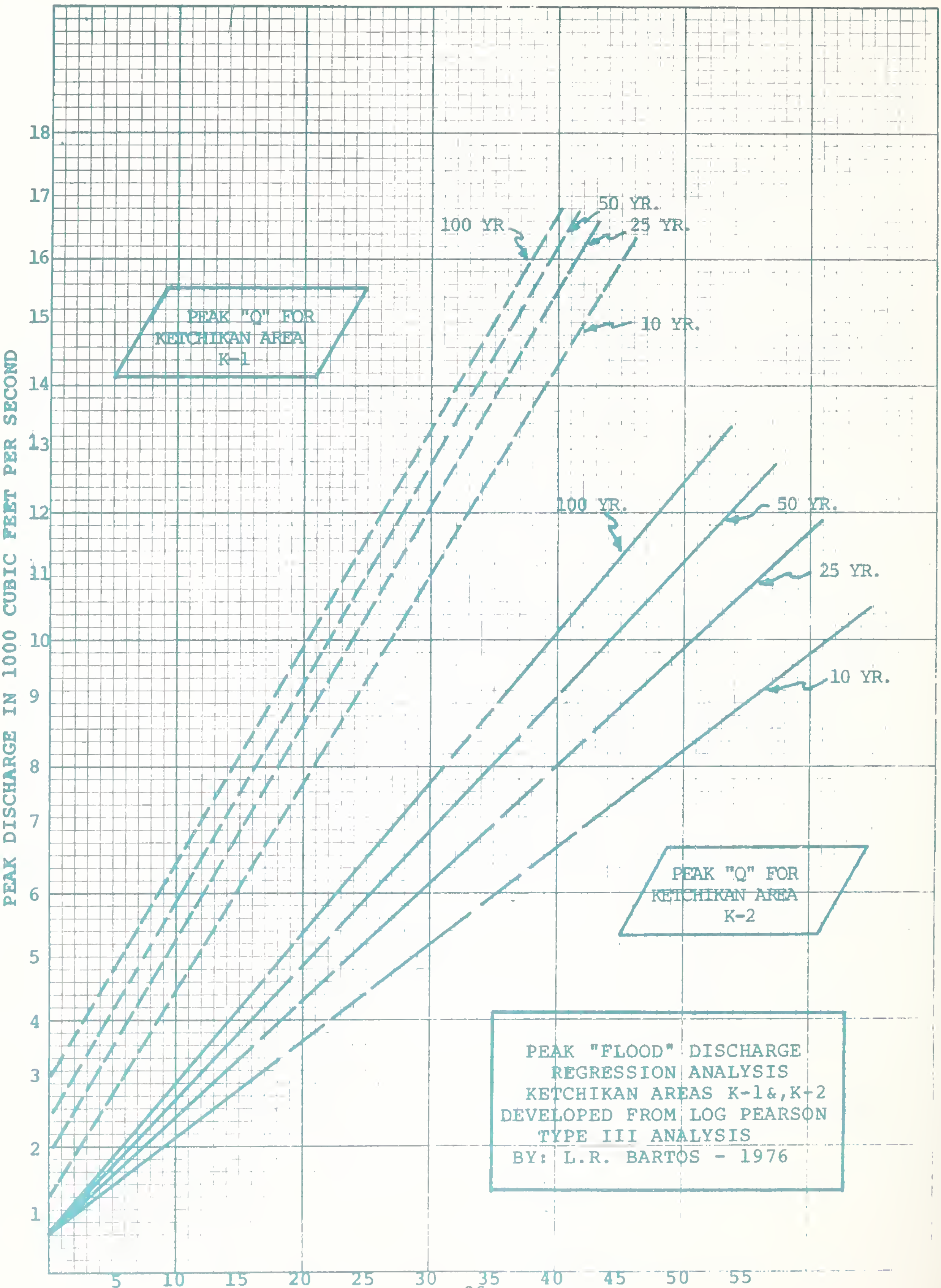
Flood-Hydrologic Response Areas of the South Tongass N.F. for Engineering Design of Structures, 10, 25, 50, & 100 Year Recurrence Intervals.

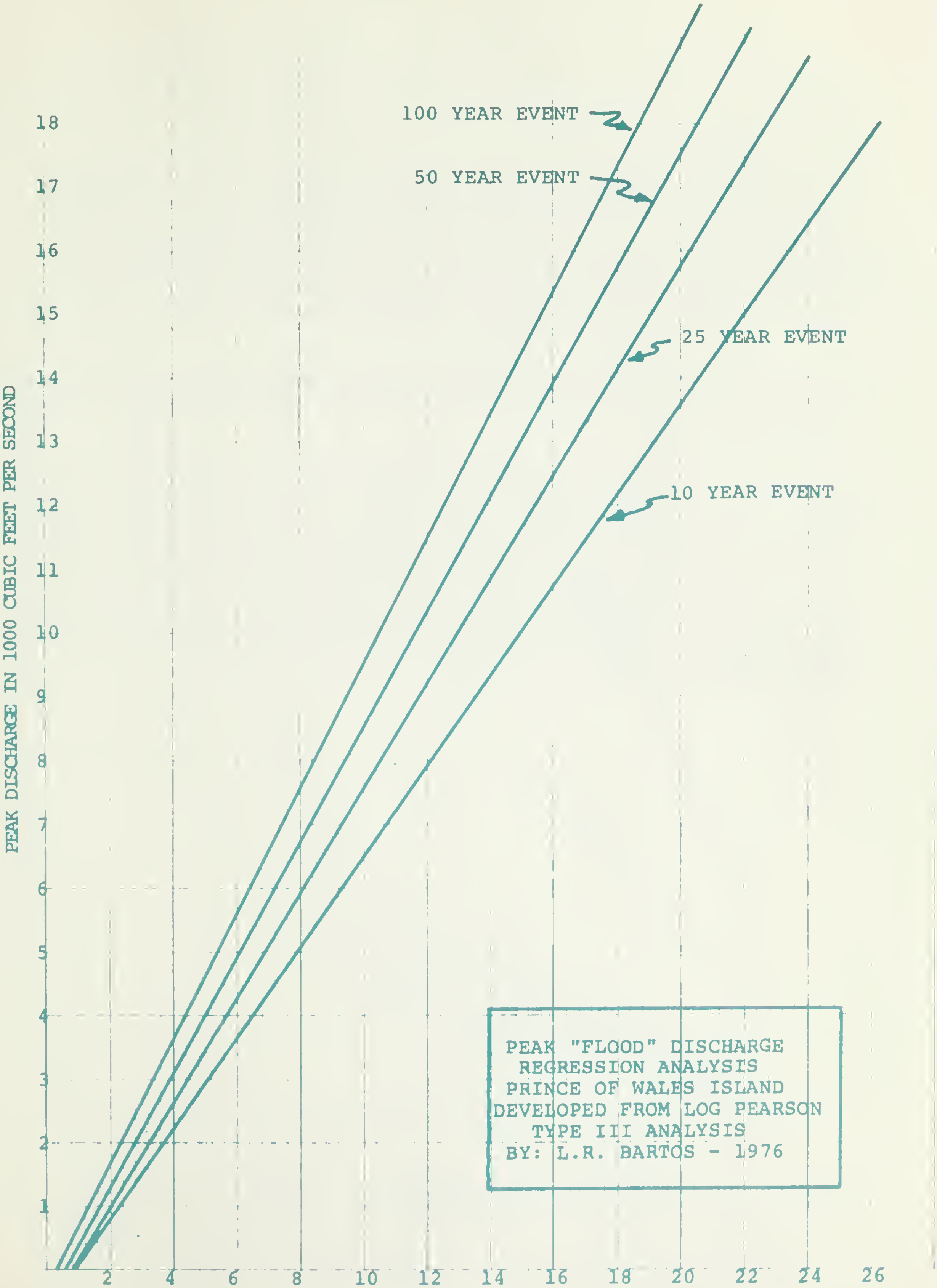


H = Drainage relief in <u>miles</u> = $H_2 - H_1$

H_R= Drainage relief factor - $\sqrt{H}$

Figure <u>1</u>   Method for determining drainage relief


PEAK DISCHARGE IN 1000 CUBIC FEET PER SECOND
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
5
10
15
20
25
30
35
40
45
50
55
PEAK "Q" FOR KETCHIKAN AREA K-1
100 YR.
50 YR.
25 YR.
10 YR.
100 YR.
50 YR.
25 YR.
10 YR.
PEAK "Q" FOR KETCHIKAN AREA K-2
PEAK "FLOOD" DISCHARGE
REGRESSION ANALYSIS
KETCHIKAN AREAS K-1&,K-2
DEVELOPED FROM LOG PEARSON
TYPE III ANALYSIS
BY: L.R. BARTOS - 1976


100 YEAR EVENT
50 YEAR EVENT
25 YEAR EVENT
10 YEAR EVENT
PEAK DISCHARGE IN 1000 CUBIC FEET PER SECOND
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
2
4
6
8
10
12
14
16
18
20
22
24
26
PEAK "FLOOD" DISCHARGE
REGRESSION ANALYSIS
PRINCE OF WALES ISLAND
DEVELOPED FROM LOG PEARSON
TYPE III ANALYSIS
BY: L.R. BARTOS - 1976

The drainage area in square miles is multiplied by the drainage relief factor in miles. With this product you enter the graph on the abscissa, go straight up to the appropriate flood recurrence interval, then go directly to the ordinate which gives the estimated peak instantaneous discharge in cubic feet per second. This can be done for any design frequency desired and is based on the new supplement in the F.S. Manual 7700.

From analysis of Stream Gage Data on Prince of Wales Island, the coefficient of the correlation for these graphs range from .88 to .91, which is good for hydrologic data.

An economical design of a drainage structure should be tempered with the desired life and staying power. This is known as "risk". This implies that it will be necessary to take some risk that a flood greater than the design flood will occur at some point during the life of the structure. This indicates that for a culvert we are willing to "risk" a 10 percent chance of failure.* For a structure based on a 10-year design life, graph no. 1, tells us that it is only necessary to use a 100-year design recurrence interval. With this information we can install a smaller diameter pipe.

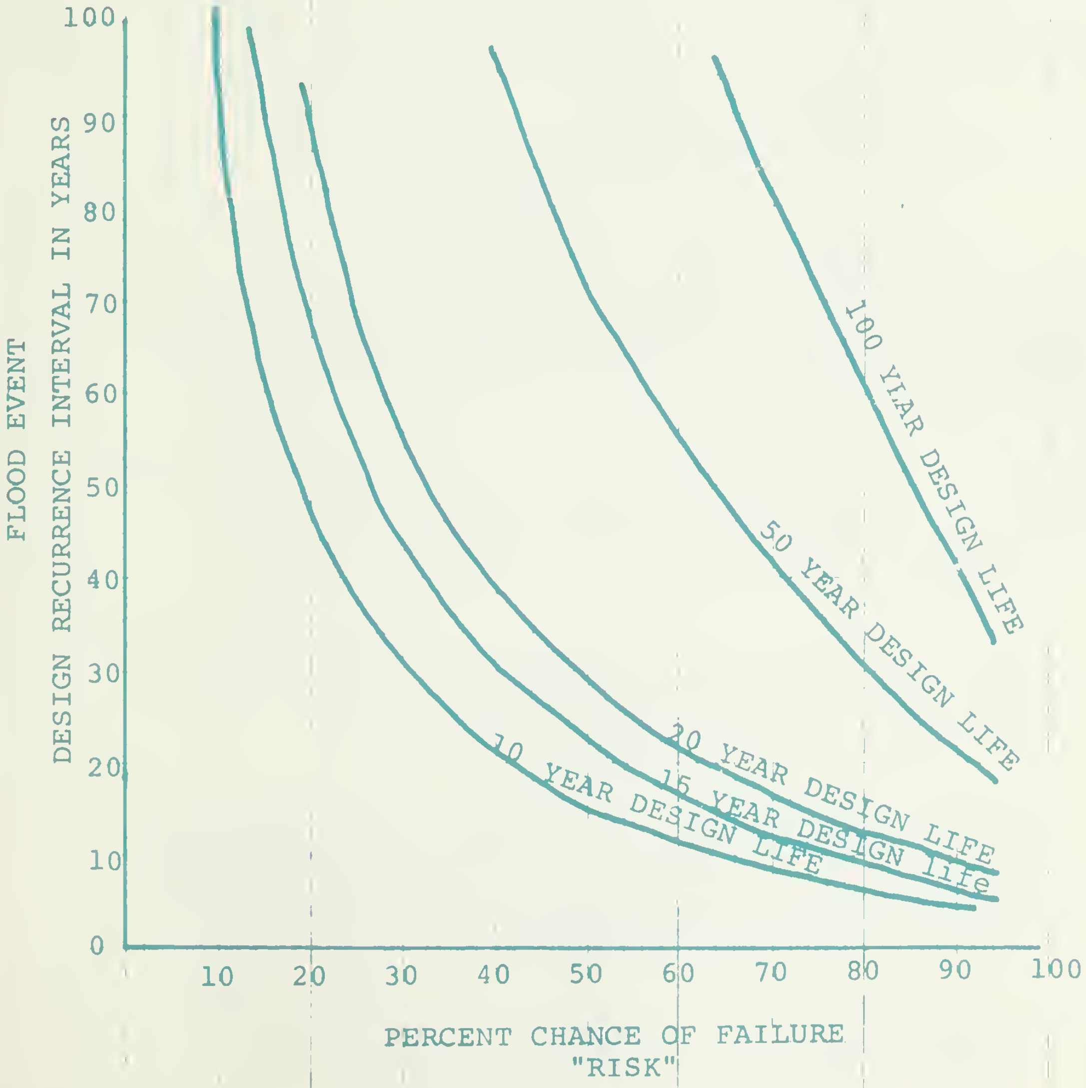
*Failure - The loss or total damage to a "structure" which requires reconstruction, due to its inability to pass safely, a design flow, or greater, of water. When such failure occurs there is also a greater damage to the downstream and adjacent environment.

DESIGN (FLOOD OR STORM) RECURRENCE INTERVALS (YEARS) NEEDED TO PROVIDE A GIVEN PROJECT LIFE WITH A GIVEN CHANCE OF FAILURE.

BASED ON FORMULA - $J = 1 - (1-{}^{1}/_{T})^{T}$, WHERE N = DESIGN LIFE, T = FLOOD REC. INT., J = CHANCE OF FAILURE.

FROM CHOW - (8-34)

BY: L.R. BARTOS - 1-14-76



On channels where a check of actual flood to graphic data is desired, and on channels with an obvious extreme high-water mark, supplemental field data are needed. The method to be used is the "Slope-Area Measurement" which is based on the Manning's equation. In the slope-area method, the discharge is computed based on the following criteria: steady, uniform flow must exist. The Manning's equation was developed for conditions of uniform flow in open channels in which the water-surface profile and energy gradient are parallel to the streambed and the area. Hydraulic radius and depth remain constant throughout the reach.* This is most important in order for the method to work for you properly and give reliable answers.

The Manning's equation is written in terms of discharge "Q".

$$Q = \frac{1.486}{N} A \quad R^{2/3} \quad S^{1/2}$$

Where

Q = Discharge in cfs
A = Cross-sectional area, ft
R = Hydraulic Radius, Ft = $\dfrac{A}{\text{wetted perimeter}}$
S = Slope or hydraulic gradient, ft/ft
N = Channel roughness coefficient

The Manning's equation was developed for conditions of uniform flow in open channels in which the water-surface profile and energy gradient are parallel to the stream bed and the area, hydraulic radius, and depth remain constant throughout the reach.

The following considerations are important in a slope-area measurement:

1. Selection of reach: A reasonably straight reach of the channel should be selected where good high-water marks can be found.

2. Cross-sectional area: One or more cross-sections should be taken for the channel and the floodplain. A simple trapezoidal channel section is most desirable; however, a compound channel can be used when properly subdivided.

3. High-water marks: Many kinds of floating materials, chiefly vegetative, and water-borne mud and silt will at times leave easily recognizable lines along the banks or on vegetation lining the bank. Careful observation of these will indicate the water level at the maximum stage. A high-water profile based on these marks should be established.

* Reach - An extended portion of water, as in a straight portion of a stream or river; a level stretch.

A more accurate determination of high-water elevation can be obtained by visual observation during the maximum stage or information received from other reliable observers.

4. Friction factor "n": The channel and the floodplain roughness coefficients should be determined using Table 1-5 as a guide. It may be necessary to subdivide the cross-section and assign different friction factors to each section.

5. Channel slope: The channel slope may be determined from U.S.G.S. topographic sheets or a channel profile.

Outline of procedure is as follows:

1. Using the typical cross-section and high-water mark the cross-sectional area of the flood flow (A) and wetted perimeter (P) can be obtained.

2. From A and P above the hydraulic radius (R) can be obtained.

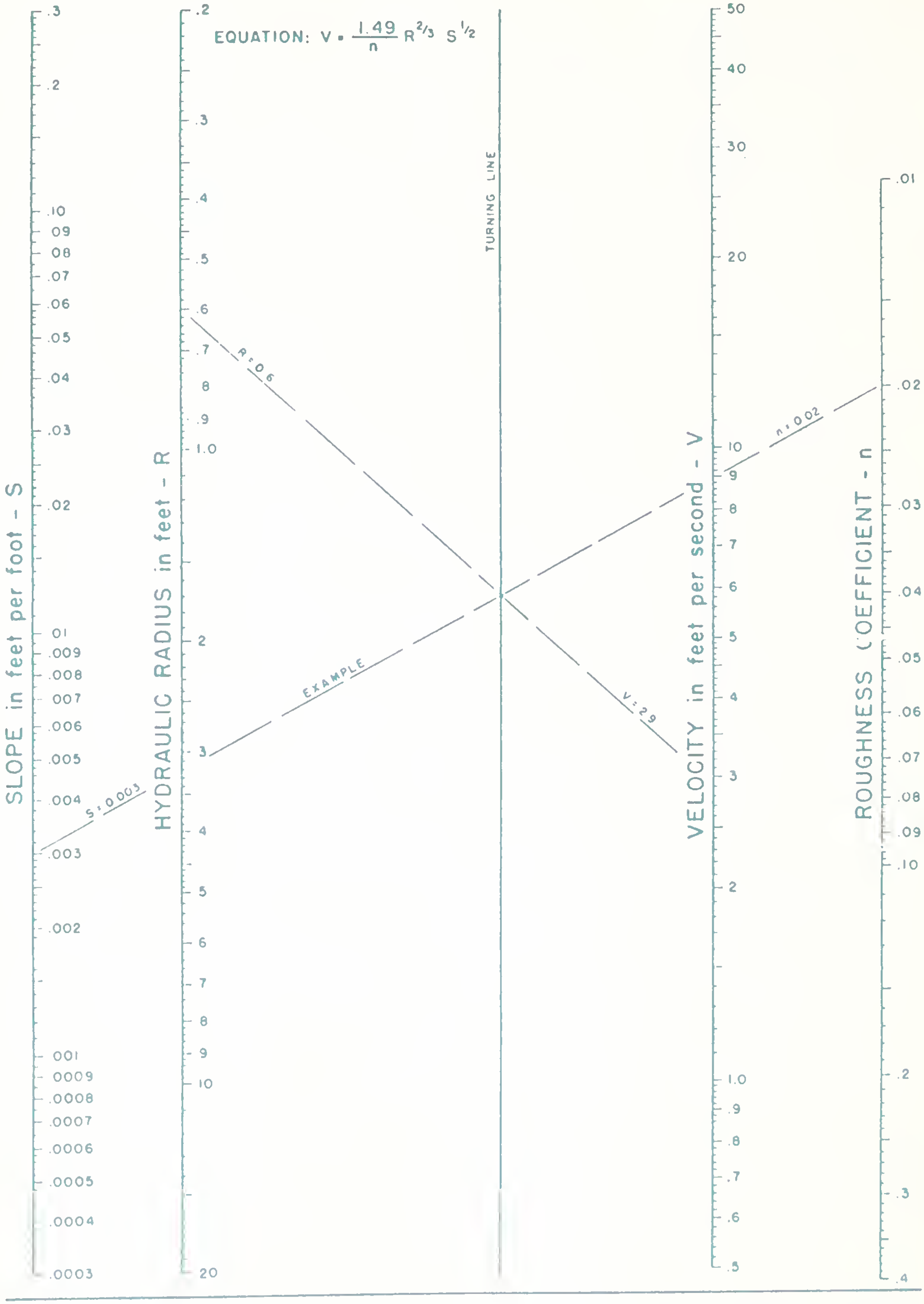
$$R - A/P$$

3. From field measurements the slope of the stream bed (S), at the design site, can be obtained in ft/ft.

4. From visual observations in the field the particle size of the bed load can be estimated and correlated to the coefficients given in Table 1-5 for the (n) value.

5. Substituting this data into Manning's equation or using the Nomograph shown in Figure 5 one obtains the flow for the given high-water mark.

See example page 17 .



NOMOGRAPH FOR SOLUTION
OF MANNING'S EQUATION

TABLE 1-5

MANNING'S ROUGHNESS COEFFICIENTS

NATURAL STREAM CHANNELS	Min	Max
I. Minor Streams		
A. Fairly regular section		
1. Some grass and weeds; little or no brush . . .	0.030	0.035
2. Dense growth or weeds, depth of flow materially greater than weed height	0.035	0.050
3. Some weeds, light brush on banks	0.035	0.050
4. Some weeds, heavy brush on banks	0.050	0.070
5. Some weeds, dense willows on banks	0.060	0.080
6. For trees within channels with branches submerged at high stage, increase all values above by	0.010	0.020
B. Irregular section with pools, slight channel meander, use A1 to A5 above, and increase all values by	0.010	0.020
C. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage		
1. Bottom: gravel, cobbles and few boulders . . .	0.040	0.050
2. Bottom: cobbles with large boulders	0.050	0.070
II. Flood Plain (adjacent to natural streams)		
A. Pasture, no brush		
1. Short grass	0.030	0.040
2. Tall grass	0.035	0.050
B. Cultivated areas		
1. No crop	0.030	0.040
2. Mature row crops	0.035	0.045
3. Mature field crops	0.040	0.050

C.	Heavy weeds, scattered brush	0.050	0.070
D.	Light brush and trees	0.060	0.080
E.	Medium to dense brush	0.100	0.160
F.	Dense willows, not bent over by current	0.150	0.200
G.	Cleared land with tree stumps 100-150 per acre		
	1. No sprouts	0.040	0.050
	2. With heavy growth of sprouts	0.060	0.080
H.	Heavy stand of timber, a few down trees, little undergrowth		
	1. Flood depth below branches	0.100	0.120
	2. Flood depth reaches branches	0.120	0.160

III. Major Streams

Roughness coefficient is usually less than for minor streams of similar description due to less effective resistance offered by irregular banks or vegetation on banks. Values of "n" for larger streams of stable or uniform sections, with no boulders or brush may be in the range from 0.028 to 0.033.

LINED CHANNELS

1.	Corrugated metal	0.021	0.024
2.	Neat cement lined	0.012	0.014
3.	Concrete	0.012	0.018
4.	Cement rubble	0.020	0.025

GRASS COVERED SMALL CHANNELS, SHALLOW DEPTH

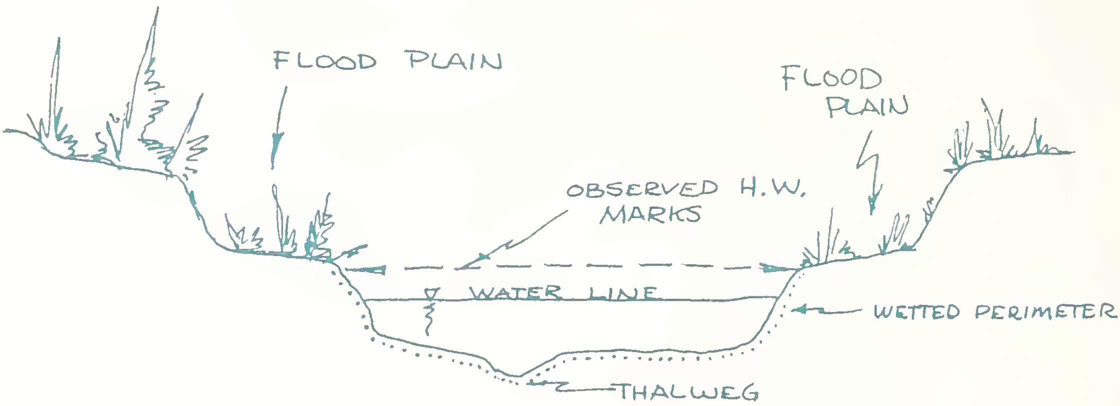
1.	No rank growth	0.035	0.045
2.	Rank growth	0.040	0.050

UNLINED CHANNELS

1.	Earth, straight and uniform	0.017	0.025
2.	Dredged	0.025	0.033
3.	Winding and sluggish	0.022	0.030
4.	Stony beds, weeds on bank	0.025	0.040

5.	Earth bottom, rubble sides	0.028	0.035
6.	Rock cuts, smooth and uniform	0.035	0.040
7.	Rock cuts, rugged and irregular	0.040	0.045
PIPE			
1.	Cast iron, coated	0.010	0.014
2.	Cast iron, uncoated	0.011	0.015
3.	Wrought iron, galvanized	0.013	0.017
4.	Wrought iron, black	0.012	0.015
5.	Steel, riveted and spiral-smooth	0.013	0.017
6.	Steel, corrugated (1/2 inch)	0.021	0.024
7.	Steel, corrugated (2 inch Structural Plate)	0.030	0.033
8.	Concrete	0.011	0.013
9.	Vitrified sewer pipe	0.012	0.014
10.	Clay, common drainage tile	0.013	0.015

EXAMPLE USING MANNING'S EQUATION



GIVEN:	X Sectional Area in ft^2	305 ft^2
	Wetted Perimeter, ft	57
	Channel Slope (for that reach)	ft/Ft. 0.01 ft/Ft.
	Channel Roughness	17 (Mannings's) = 0.040

REQUIRED: Peak Discharge in c.f.s.

1. Hydraulic Radius = $\frac{\text{Area}}{\text{Wetted Perimeter}}$

2. Peak Discharge = $\frac{1.49}{17}$ A $R^{0.6666}S^{0.5}$

$$= \frac{1.49}{0.040} (305)\ (5.35)^{0.6666} (0.01)^{0.5}$$

$$= 3475 \text{ cfs}$$

3. Velocity: $Q = AV$

$$V = \frac{Q}{A} = \frac{3475}{305}$$

$$V = 11.39 \text{ f.p.s.}$$

Minimum Flows for Fish Passage

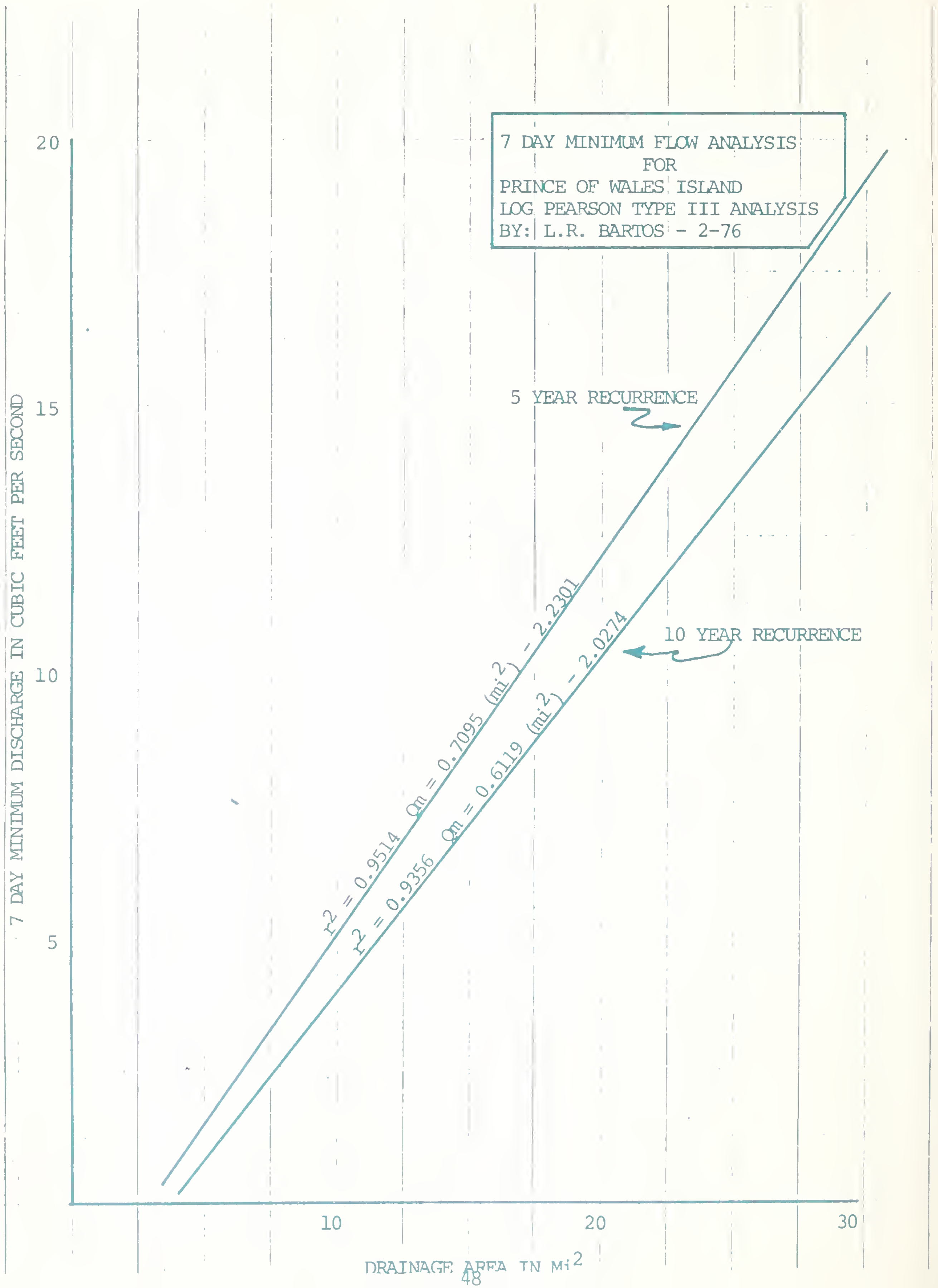
The minimum flows are included for the purpose of passing small fish through culverts. To obtain the critical minimum flow, average the 7 consecutive days of lowest flows of a water year. Fisheries has determined that the 5-year recurrence in years is sufficiently low. Lower flows would cause fish to become stranded or remain in deeper pools until flows increase.

The method in obtaining the minimum expected flow during a 7-day minimum, is basically the same as that done previously with maximum discharge, using graph 7, enter with drainage area to the desired recurrence interval thence to the discharge in cubic feet per second.

Consideration which should be made when designing these culverts for the passage of fish is:

1. Minimize the grade of the culvert as much as possible, without developing a sediment-bed load hazard problem.

2. Insure that the design prevents "shot gun" or dropoffs at the tail water of the culvert thus creating a fish barrier.


7 DAY MINIMUM FLOW ANALYSIS
FOR
PRINCE OF WALES ISLAND
LOG PEARSON TYPE III ANALYSIS
BY: L.R. BARTOS - 2-76
5 YEAR RECURRENCE
10 YEAR RECURRENCE
r2 = 0.9514 Qm = 0.7095 (mi2) - 2.2301
r2 = 0.9356 Qm = 0.6119 (mi2) - 2.0274
7 DAY MINIMUM DISCHARGE IN CUBIC FEET PER SECOND
20
15
10
5
10
20
30
DRAINAGE AREA IN Mi2

A GUIDE TO REGIONALIZATION OF HYDROLOGIC DATA FOR DESIGN ON UNGAGED WATERSHEDS IN SOUTHEAST ALASKA

By L.R. Bartos

OBJECTIVE

The primary use of this document is to permit the engineer to evaluate a drainage for a particular precipitation runoff event without the use of long, drawn out manipulative procedures. The investigator needs only drainage area and mean drainage slope to be able to determine a design flood peak in cubic feet per second, c.f.s.

METHODS

The method used was basically that of taking existing U.S.G.S. published records and applying regression analysis approach to the data. In most cases the basin area in square miles is the independent parameter and can be easily obtained.

It should be noted that information published herein has inherent limitations, therefore, don't extrapolate the data beyond its intent.

Expansion of the "Design Guide for Determination of Design Flow for Hydraulic Structures in Southeast Alaska" should be as follows:

The Region and the Forests are now supplied with printouts of the updated Log Pearson Type III Statistical Analysis for Flood Frequency and a flow duration analysis for all U.S.G.S. gaging stations with ten or more years of data. With this data many evaluations can be made and presented in a usable form.

The basic approach to the regional flood peak discharge for bridge and culvert design is to obtain Log Pearson Type III Flood Probability Analysis printouts from the U.S.G.S. The data is printed in a tabular format (Fig. 1a) with the drainage area (D.A.) for a given site. These stations are then tabulated by station name and number, area in square miles. From the Log Pearson printouts, Figs. 1a, 2a, and 3a, pick off the 10, 25, 50, and if needed, the 100-year recurrence interval flood flows and tabulate, as shown in Fig. 1.

Drainage relief is the parameter that must be determined from topographic maps. The method for determining drainage relief is given in Fig. 1. These data are then regressed, the "drainage area x relief factor" being the independent variable and flood discharge being the dependent variable. It should be noted that these points could be plotted on an appropriate graph paper prior to the regression analysis, to get an indication of probable correlation or separation.

When the individual points are plotted they should be accompanied by an appropriate number, for example, 1, 2, 3, 4,.........n, as they are listed on the tabular sheets. Upon separation, the numbers can be checked against the stations for geographic similarity. This is important for the determination of hydrologic response areas.

The data from the estimated best fit separations can then be handled in two ways:

1. Desk top calculator statistical analysis. An estimate can be made as to the best fit line, i.e, linear, exponential, hyperbolic, etc., then run the appropriate programs on a programmable desk top calculator.

2. UNIVAC 1100, through Ft. Collins, and the use of the program in BASIC STAT., OLD:CURFIT. In using this program the user inputs the independent and dependent variables on a time share terminal and the output is instantaneous. The data is analyzed in 6 different curve types, along with a corresponding correlation coefficient "r^2". The operator can then select the curve giving the highest r^2 and receive an expanded analysis of the best curve fit.

It is most important that a coefficient of correlation r "2" be run since this gives you a measure of strength of this analysis. If within a geographic unit there is a point that is a very poor fit, or a "flier", the background and basic data should be researched to determine the cause for erratic data. The problem could stem from short term of data, poor data (as noted under remarks), glacier dam outwash, or perhaps intense timber harvest within the drainage. Perhaps this point, for one of those reasons previously cited can be thrown out of the data set, which will improve the correlation coefficient of the curve.

Another way to improve the correlation is to make adjustments in the initial area stratification and go through the analysis process once again.

When a proposed bridge or culvert is some distance up stream from a stream gaging station, the following equation can be used to determine an adjusted discharge:

$$Q_1 A_1^{-0.5} = Q_2 A_2^{-0.5}$$

Q_1 = Discharge at gaging station

A_1 = Drainage Area above gaging station

Q_2 = Unknown discharge at construction site

A_2 = Drainage Area above construction site

Figure 1a

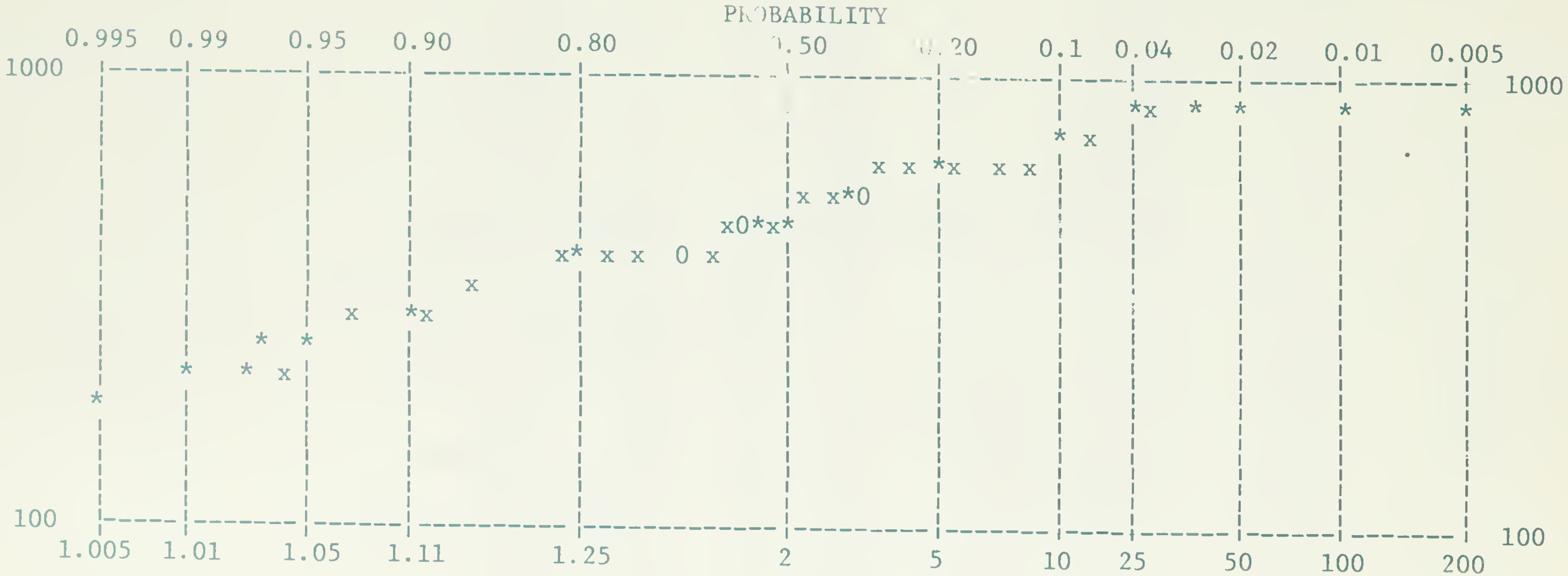
STATION 15060000 PERSEVERANCE C NR WACKER AK

TOTAL D.A. = 2.81 CONTR. D.A. =
GAGE DATUM = 600. FT.

WATER YEAR	ANNUAL PEAK DISCH,CFS	DATE	CODES	HIGHEST SINCE	GAGE HEIGHT OF ANNUAL PEAK,FT	CODE	ANNUAL MAX GAGE HT,FT	DATE	CODE
1932	0000359	02-23-32							
1938	0000420	10-22-37							
1939	0000213	09-18-39							
1947	0000506	10-06-46			5.23				
1948	0000265	10-14-47			3.88				
1949	0000339	04-26-49			4.33				
1950	0000543	10-30-49			5.26				
1951	0000409	06-11-51			4.51				
1952	0000356	10-07-51			4.20				
1953	0000344	05-12-53			4.13				
1954	0000492	02-02-54			4.98				
1955	0000447	10-16-54			4.73				
1956	0000380	10-23-55			4.34				
1957	0000387	12-25-56			4.38				
1958	0000505	04-11-58			5.05				
1959	0000510	10-21-58			5.08				
1960	0000451	12-05-59			4.77				
1961	0000557	10-22-60			5.34				
1962	0000530	10-14-61			5.19				
1963	0000496	12-10-62			5.00				
1964	0000295	12-30-63			3.84				
1965	0000685	10-18-64			5.68				
1966	0000656	03-29-66			5.57				
1967	0000400	08-09-67			4.40				
1968	0000493	10-09-67			4.84				
1969	0000331	11-19-68			4.04				

PEAKS MARKED WITH * NOT PASSED TO LOG PEARSON PROGRAM

PERSEVERANCE C NP WACKER AK 1932- NO. OF ITEMS = 26 STATION 15- 600.0 CODE PK



THE FOLLOWING SYMBOLS MAY APPEAR IN THE PLOT
x - AN INPUT DATA VALUE
* - A CALCULATED VALUE
0 - A CALCULATED VALUE AND ONE DATA VALUE AT SAME POSITION
2 - TWO INPUT DATA VALUES PLOTTED AT SAME POSITION
3 - THREE INPUT DATA VALUES PLOTTED AT SAME POSITION
A - A CALCULATED VALUE AND TWO DATA VALUES AT THE SAME POSITION
B - A CALCULATED VALUE AND THREE DATA VALUES AT THE SAME POSITION

Figure 2a

Figure 3a

PERSEVERANCE C NR WACKER AK 1932-69 NO. OF ITEMS = 26 STATION 15 - 600.0 CODE PK

D A T A U S E D I N C A L C U L A T I O N S

359.000	420.000	213.000	506.000	265.000	339.000	543.000	409.000	356.000	344.000
492.000	447.000	380.000	387.000	505.000	510.000	451.000	557.000	530.000	496.000
295.000	685.000	656.000	400.000	493.000	331.000				

A N N U A L F L O O D S T A T I S T I C S

	LOGS	CFS
MEAN =	2.626	437.3
STANDARD DEVIATION =	0.118	112.9
SKEWNESS =	-0.520	0.219
STANDARD ERROR OF SKEWNESS =	0.456	

L O G - P E A R S O N T Y P E I I I C A L C U L A T I O N S

EXCEEDANCE PROB	RECURRENCE INTERVAL	MAGNITUDES
0.9900	1.01	203.094
0.9500	1.05	260.778
0.9000	1.11	295.040
0.8000	1.25	339.560
0.5000	2.00	432.701
0.2000	5.00	533.352
0.1000	10.00	587.523
0.0400	25.00	645.482
0.0200	50.00	682.644
0.0100	100.00	715.667 *** R.I. > 2N
0.0050	200.00	745.417 *** R.I. > 2N
0.0020	500.00	780.698 *** R.I. > 2N

APPENDIX

DESIGN

The design flow for which the structure is to designed should be determined from one of the following methods using the design frequency determined above. They are listed in order of reliability. The methods probably most commonly applicable to Forest Service structures are B and C.

A. Log-Pearson Type III Method - is a statistical analysis of the actual stream gage data for a specific stream. This method is not feasible for most R-10 Forest Service structures due to the limited amount of stream gage data available. If such data is available it should be used according to the procedure outlined in Appendix A. Publication 2 listed in the introduction contains results of such analysis on many of the major drainages in Southeast Alaska and may be used if applicable.

B. U.S.G.S. Regional Flood Frequency "Equation" for Alaska - is the method probably most applicable for Forest Service Structures in Alaska. It is based on a regression analysis of the stream gage data collected through 1970 in Alaska. Publication 2 discusses this method and its basis in detail. The general procedure is outlined in Appendix B.

C. U.S.G.S. Regional Flood Frequency Curved for Southeast Alaska is a method presented in 1964 by the U.S.G.S. in Circular No. 493. This procedure is outlined in Appendix C. It is their initial attempt at trying to analyze the stream gage data, however, consider only the drainage basin area as an independent parameter. Method B, above, is merely a refinement of this method which incorporates more of the parameters known to effect runoff.

For example problem see Appendix Z.

APPENDIX A

Log Pearson Type III Method

The following information was extracted from the Water Resources Council Bulletin No. 15, entitled "A Uniform Technique for Determining Flood Flow Frequencies" dated December 1967.

The Pearson Type III Method, was originally presented by H.A. Foster in 1924. As used by Foster, the method required the use of the natural data in computations of the mean, standard deviation, and skew coefficient of the distribution. The current practice, is first to transform the natural data to their logarithms and then to compute the statistical parameters. Because of the transformation the method is now called the Log-Pearson Type III Method.

Outline of the Method

The following symbols are used in the outline of Flood Flow Frequency Analysis, which is based on the presentation in Bulletin 13.

Y = Arithmetic magnitude of an annual flood event.

X = Logarithmic magnitude of Y (Natural Log).

N = Number of events in the record being used.

M = Mean of the X's.

$x = X-M$

S = Standard deviation of the X's.

g = Skew coefficient.

K = Pearson Type III coordinates expressed in number of standard deviations from the mean for various recurrence intervals or percent chance.

Q = Computed flood flow for a selected recurrence interval or percent chance.

The events considered here are flood flows in the annual series. (Definitions of hydrological and statistical terms used here are found in the Glossary of Bulletin 13). In the work, the physical units used for Y (such as cfs) are also those for Q.

The outline of work is as follows:

1. Transform the list of N annual flood magnitudes Y_1, $Y_2, \ldots\ldots, Y_N$ to a list of corresponding logarithmic magnitudes $X_1, X_2, \ldots\ldots, X_N$.

2. Compute the mean of the logarithms:

$$M = \frac{\sum X}{N}$$

3. Compute the standard deviation of the logarithms:

$$S = \sqrt{\frac{\sum x^2}{N - 1}}$$

$$= \sqrt{\frac{\sum X^2 - (\sum X)^2 / N}{N - 1}}$$

4. Compute the coefficient of skewness:

$$g = \frac{N \sum x^3}{(N - 1)(N - 2)S^3}$$

$$= \frac{N^2 \sum X^3 - 3N \sum X \sum X^2 + 2(\sum X)^3}{N(N - 1)(N - 2)S^3}$$

5. Compute the logarithms of discharges at selected recurrence intervals or percent chance:

$$\log Q = M + K\,S$$

Take K from Table 1 or Table 2 for the computed value of g and the selected recurrence interval or percent chance. Log Q is the logarithm of a flood discharge having the same recurrence interval or percent chance.

6. Find the antilog of log Q to get the flood discharge Q.1/

Tables of K values

Tables 1 and 2 were made from larger and more complete tables prepared by H. Leon Harter (Mathematical Statistician, Wright-Patterson Air Force Base) and the U.S. Soil Conservation Service. Copies of those tables are available, free of charge, from the Central Technical Unit, Soil Conservation Service, 269 Federal Center Building, Hyattsville, Md. 20782.

Computer Program Sources

Federal agencies such as the Bureau of Reclamation, Corps of Engineers, Geological Survey, Soil Conservation Service, Tennessee Valley Authority, and others, have prepared computer programs for the Log-Pearson Type III method. These programs are in various computer languages and for various types of computers. Inquiries regarding these programs should be addressed to those agencies.

References

(1) "Theoretical Frequency Curves," by H.A. Foster: American Society of Civil Engineers, Transactions, v. 87, p. 142-203: 1924.

(2) "Methods of Flow Frequency Analysis," by the Subcommittee on Hydrology. Inter-Agency Committee on Water Resources: Notes on Hydrologic Activities, Bulletin 13, April 1966. For sale by the Superintendent of Documents, Government Printing Office, Washington, D.C. 20402. Price 35 cents.

1/ The frequency line can be shown by plotting each Q versus its respective percent chance on lognormal probability paper and drawing a continuous line through the plotted points.

<u>Table 1 -- K values for positive skew coefficients</u>

Skew Coefficient (g)	Recurrence Interval in Years: 1.0101	1.0526	1.1111	1.2500	2	5	10	25	50	100	200
	Percent Chance: 99	95	90	80	50	20	10	4	2	1	0.5
3.0	-0.667	-0.665	-0.660	-0.636	-0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	-0.690	-0.688	-0.681	-0.652	-0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	-0.714	-0.711	-0.702	-0.666	-0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	-0.740	-0.736	-0.724	-0.681	-0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	-0.769	-0.762	-0.747	-0.696	-0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	-0.799	-0.790	-0.771	-0.711	-0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	-0.832	-0.819	-0.795	-0.725	-0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	-0.867	-0.850	-0.819	-0.739	-0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	-0.905	-0.882	-0.844	-0.752	-0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	-0.946	-0.914	-0.869	-0.765	-0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	-0.990	-0.949	-0.895	-0.777	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	-1.037	-0.984	-0.920	-0.788	-0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	-1.087	-1.020	-0.945	-0.799	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	-1.140	-1.056	-0.970	-0.808	-0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	-1.197	-1.093	-0.994	-0.817	-0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	-1.256	-1.131	-1.018	-0.825	-0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	-1.318	-1.168	-1.041	-0.832	-0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	-1.383	-1.206	-1.064	-0.838	-0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	-1.449	-1.243	-1.086	-0.844	-0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	-1.518	-1.280	-1.107	-0.848	-0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	-1.588	-1.317	-1.128	-0.852	-0.164	0.758	1.340	2.043	2.542	3.022	3.489
.9	-1.660	-1.353	-1.147	-0.854	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
.8	-1.733	-1.388	-1.166	-0.856	-0.132	0.780	1.336	1.993	2.453	2.891	3.312
.7	-1.806	-1.423	-1.183	-0.857	-0.116	0.790	1.333	1.967	2.407	2.824	3.223
.6	-1.880	-1.258	-1.200	-0.857	-0.099	0.800	1.328	1.939	2.359	2.755	3.132
.5	-1.955	-1.291	-1.216	-0.856	-0.083	0.808	1.323	1.910	2.311	2.686	3.041
.4	-2.029	-1.524	-1.231	-0.855	-0.066	0.816	1.317	1.880	2.261	2.615	2.949
.3	-2.104	-1.555	-1.245	-0.853	-0.050	0.824	1.309	1.849	2.211	2.544	2.856
.2	-2.178	-1.586	-1.258	-0.850	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
.1	-2.252	-1.616	-1.270	-0.846	-0.017	0.836	1.292	1.785	2.107	2.400	2.670
0	-2.326	-1.645	-1.282	-0.842	0	0.842	1.282	1.751	2.054	2.326	2.576

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Table 2.--K values for negative skew coefficients

Skew Coefficient (g)	Recurrence Interval in Years: 1.0101	1.0526	1.1111	1.2500	2	5	10	25	50	100	200
	Percent Chance: 99	95	90	80	50	20	10	4	2	1	0.5
0	-2.326	-1.645	-1.282	-0.842	0	0.842	1.282	1.751	2.054	2.326	2.576
- .1	-2.400	-1.673	-1.292	-0.836	0.017	0.846	1.270	1.716	2.000	2.252	2.482
- .2	-2.472	-1.700	-1.301	-0.830	0.033	0.850	1.258	1.680	1.945	2.178	2.388
- .3	-2.544	-1.726	-1.309	-0.824	0.050	0.853	1.245	1.643	1.890	2.104	2.294
- .4	-2.615	-1.750	-1.317	-0.816	0.066	0.855	1.231	1.606	1.834	2.029	2.201
- .5	-2.686	-1.774	-1.323	-0.808	0.083	0.856	1.216	1.567	1.777	1.955	2.108
- .6	-2.755	-1.797	-1.328	-0.800	0.099	0.857	1.200	1.528	1.720	1.880	2.016
- .7	-2.824	-1.819	-1.333	-0.790	0.116	0.857	1.183	1.488	1.663	1.806	1.926
- .8	-2.891	-1.839	-1.336	-0.780	0.132	0.856	1.166	1.448	1.606	1.733	1.837
- .9	-2.957	-1.858	-1.339	-0.769	0.148	0.854	1.147	1.407	1.549	1.660	1.749
-1.0	-3.022	-1.877	-1.340	-0.758	0.164	0.852	1.128	1.366	1.492	1.588	1.664
-1.1	-3.087	-1.894	-1.341	-0.745	0.180	0.848	1.107	1.324	1.435	1.518	1.581
-1.2	-3.149	-1.910	-1.340	-0.732	0.195	0.844	1.086	1.282	1.379	1.449	1.501
-1.3	-3.211	-1.925	-1.339	-0.719	0.210	0.838	1.064	1.240	1.324	1.383	1.424
-1.4	-3.271	-1.938	-1.337	-0.705	0.225	0.832	1.041	1.198	1.270	1.318	1.351
-1.5	-3.330	-1.951	-1.333	-0.690	0.240	0.825	1.018	1.157	1.217	1.256	1.282
-1.6	-3.388	-1.962	-1.329	-0.675	0.254	0.817	0.994	1.116	1.166	1.197	1.216
-1.7	-3.444	-1.972	-1.324	-0.660	0.268	0.808	0.970	1.075	1.116	1.140	1.155
-1.8	-3.499	-1.981	-1.318	-0.643	0.282	0.799	0.945	1.035	1.069	1.087	1.097
-1.9	-3.553	-1.989	-1.310	-0.627	0.294	0.788	0.920	0.996	1.023	1.037	1.044
-2.0	-3.605	-1.996	-1.302	-0.609	0.307	0.777	0.895	0.959	0.980	0.990	0.995
-2.1	-3.656	-2.001	-1.294	-0.592	0.319	0.765	0.869	0.923	0.939	0.946	0.949
-2.2	-3.705	-2.006	-1.284	-0.574	0.330	0.752	0.844	0.888	0.900	0.905	0.907
-2.3	-3.753	-2.009	-1.274	-0.555	0.341	0.739	0.819	0.855	0.864	0.867	0.869
-2.4	-3.800	-2.011	-1.262	-0.537	0.351	0.725	0.795	0.823	0.830	0.832	0.833
-2.5	-3.845	-2.012	-1.250	-0.518	0.360	0.711	0.771	0.793	0.798	0.799	0.800
-2.6	-3.889	-2.013	-1.238	-0.499	0.368	0.696	0.747	0.764	0.768	0.769	0.769
-2.7	-3.932	-2.012	-1.224	-0.479	0.376	0.681	0.724	0.738	0.740	0.740	0.741
-2.8	-3.973	-2.010	-1.210	-0.460	0.384	0.666	0.702	0.712	0.714	0.714	0.714
-2.9	-4.013	-2.007	-1.195	-0.440	0.390	0.651	0.681	0.683	0.689	0.690	0.690
-3.0	-4.051	-2.003	-1.180	-0.420	0.396	0.636	0.660	0.666	0.666	0.667	0.667

APPENDIX B

The Water Resources Division of the United States Geological Survey, Published a report in 1970 - "Flood Frequency in Alaska", by J.M. Childers. The following design flow expressions are from that report and incorporate the parameters which were determined statistically to have an effect on runoff. Although all parameters of influence may not have been incorporated, it is felt this is the best rational approach to the problem at this time except for Method A.

2-year flood	$Q2 = 1.99\ (A)^{.90}\ (S_t+1)^{-.24}\ (p)^{.74}\ (I)^{.53}$
5-year flood	$Q5 = 3.92\ (A)^{.87}\ (S_t+1)^{-.25}\ (p)^{.66}\ (I)^{.60}$
10-year flood	$Q10 = 5.517\ (A)^{.86}\ (S_t+1)^{-.26}\ (p)^{.61}\ (I)^{.65}$
25-year flood	$Q25 = 9.25\ (A)^{.85}\ (S_t+1)^{-.35}\ (p)^{.53}\ (I)^{.81}$
50-year flood	$Q50 = 14\ (A)^{.75}\ (S_t+1)^{-.20}\ (p)^{.76}$

Where:

Q = Flow in cubic feet per sec for x year flood.

A = Drainage area in square miles for the location in question.

S_t = Area of lakes and ponds in % of drainage area.

P = Mean annual precipitation as determined from U.S. weather bureau data.

I = Precipitation intensity in inches from U.S. weather bureau data.

NOTE: Above equations have a standard error of estimate ranging from 80% to 53%. (SEE 1970 report)

Outline of procedure is as follows:

1. Using the appropriate U.S.G.S. quadrangle map one can outline the drainage area of concern.

2. Using a planimeter one can compute the area of the drainage basin A in square miles.

3. Using a quadrangle map or aerial photos of the drainage basin one can estimate the percent of lakes and ponds - S_t by % of drainage area.

For an example problem see Appendix Z, pages 5 and 6.

4. Using attached figure 2, page 8, one can obtain the mean annual precipitations in inches for the drainage basin - P in inches.

5. Using attached figure 3, page 9, one can obtain the precipitation intensity for the drainage basin - I in inches.

6. Using the appropriate X-year flood equation one can substitute these parameters and compute the anticipated design flow.

For an example problem see Appendix Z, pages 2 to 5.

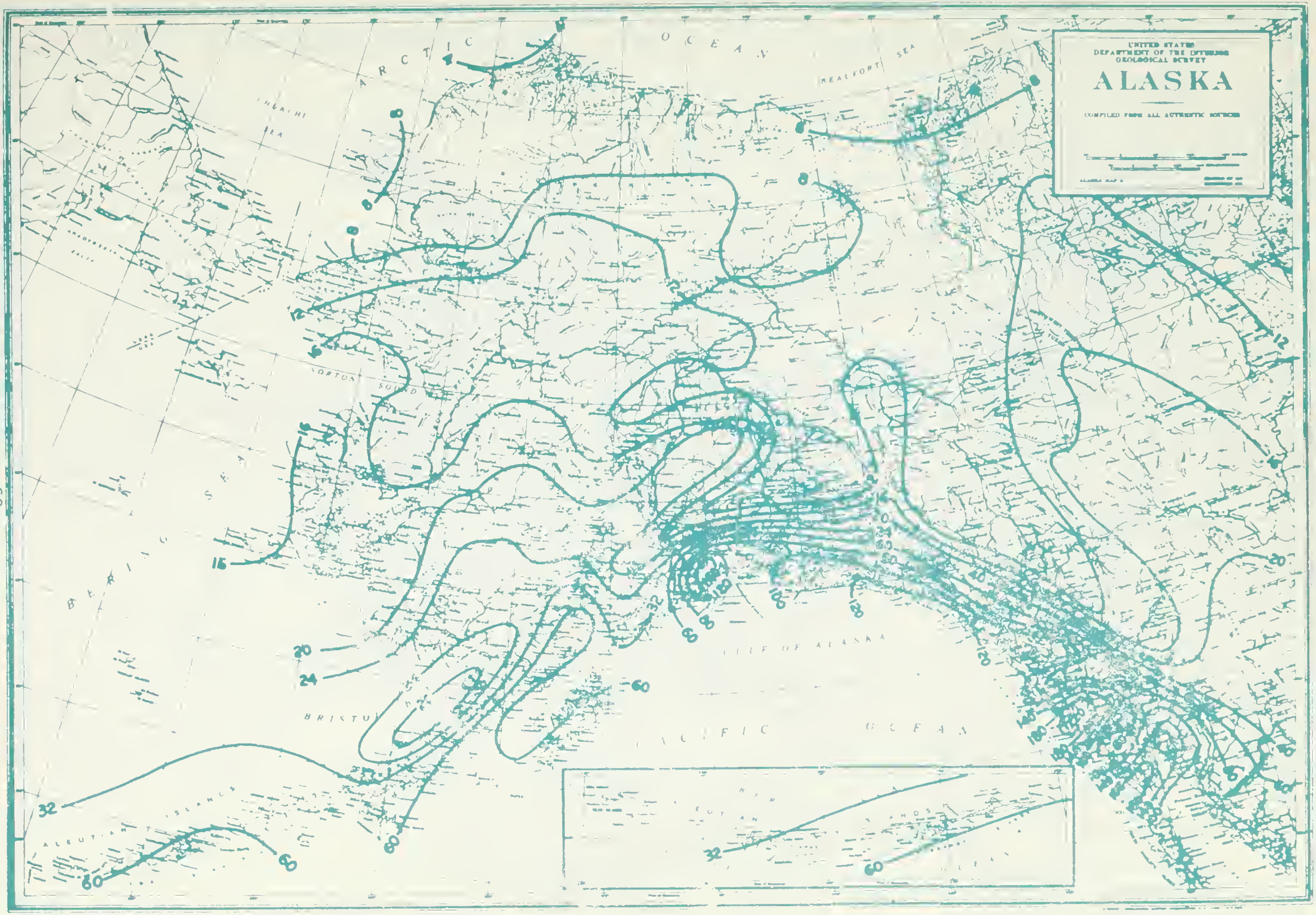


Figure 2.--Mean annual pre ipitat on in inches (1931-1955).

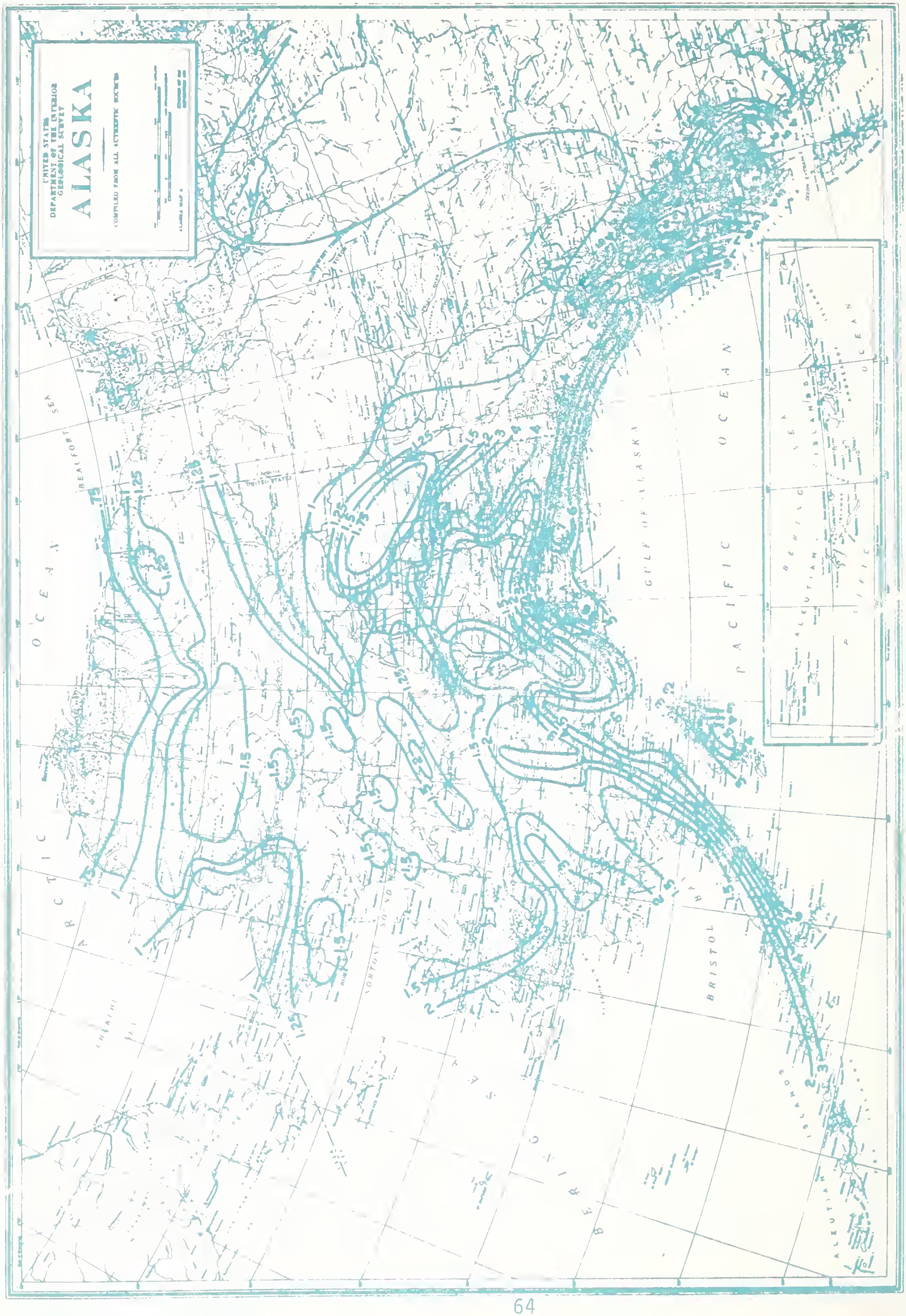


Figure 3.--Two-year 24-hour rainfall in inches.

APPENDIX C

:he Geological Survey published Circular No. 493, "Magnitude and Frequency of flood in Alaska, South of the Yukon River" in 1964. The information below is from that report and reflects the general relationship of Drainage Area versus Flood Discharge for Southeast Alaska. The shaded envelope was added and indicates the general maximum and minimum flood flows for the available data.

Outline of Procedure

1. Using the appropriate U.S.G.S. quadrangle map one can outline the drainage area of concern.

2. Using a planimeter one can compute the drainage basin area, A in square miles.

3. Referring to Figure 4, Page 11, one can, by direct reading or interpolation, relate the anticipated flood discharge to the drainage basin area.

For example problem see Appendix Z, page 6.

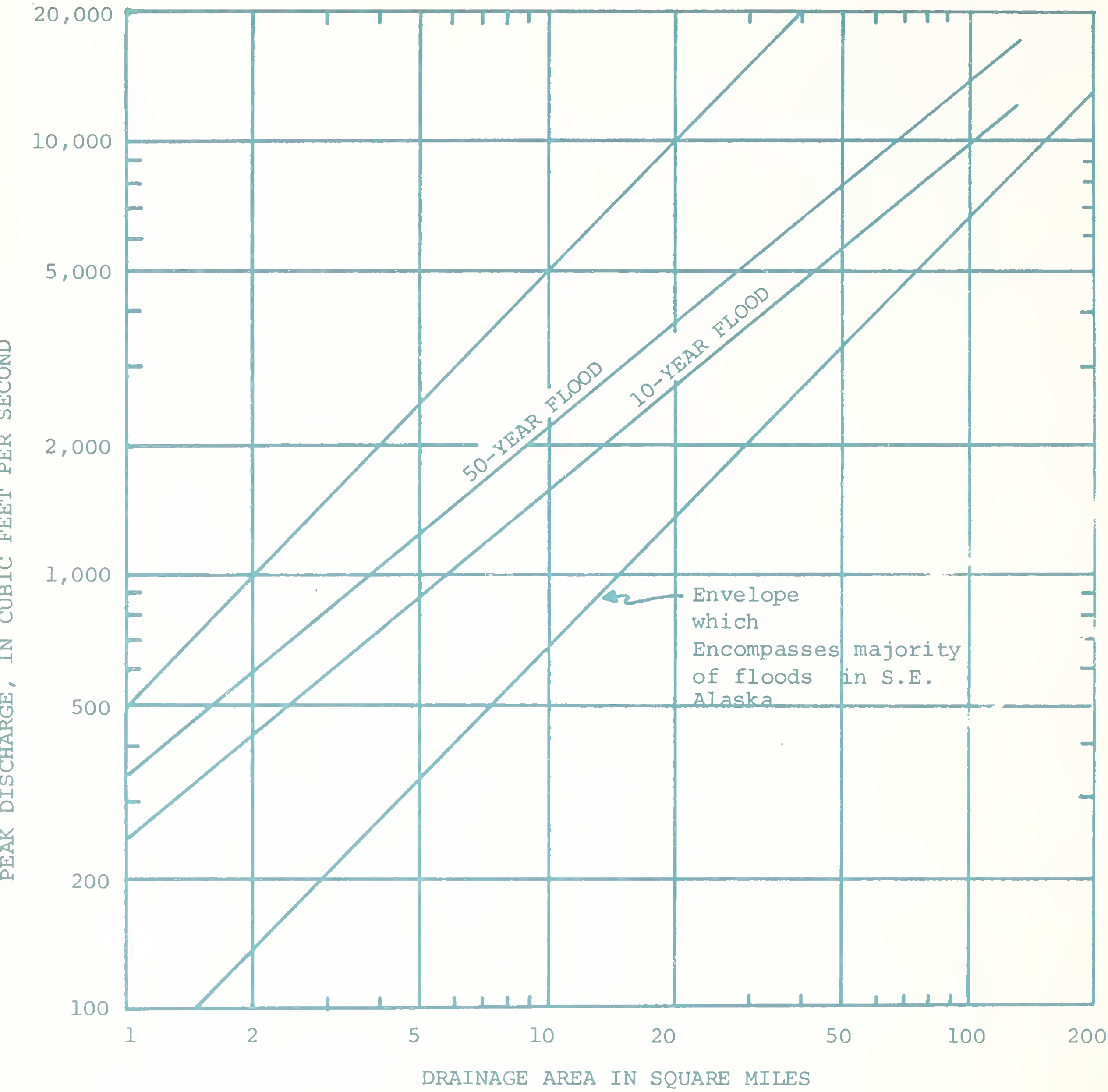


Figure 4

10- AND 50- YEAR FLOOD IN HYDROLOGIC AREA

APPENDIX - Z

Example Problem

A. Given:

Proposed permanent bridge is to be constructed across Maybeso Creek at Hollis in the South Tongass National Forest. Bridge will have about a 75' clear span.

a. Gage Station Records over the past 15 years recorded the following data:

Year	Peak Discharge (CFS)
1949	2110
1950	3270
1951	1320
1952	2460
1953	2010
1954	1550
1955	1500
1956	1940
1957	2430
1958	1650
1959	2510
1960	3430
1961	1700
1962	3760
1963	3000

b. Review of the U.S.G.S. Map and Aerial Photo of the drainage basin indicate the following characteristics:

(S_t) Area of lakes and ponds - 0%

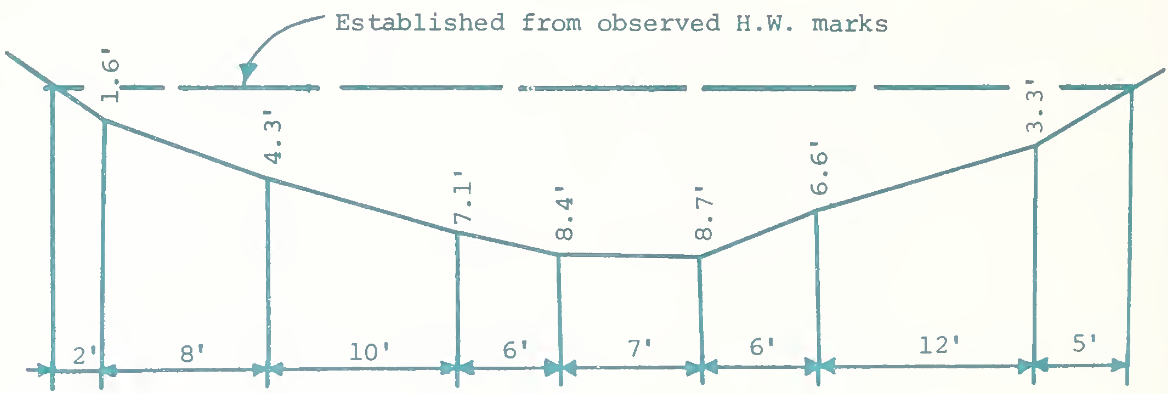
Using a planimeter on the U.S.G.S. quadrangle map we find:

(A) Drainage Area - 15.1 sq. miles

c. Field measurements at the proposed site yield the following data:

Stream bed is composed of 4" minus gravels and has an average slope through the site area of .005 ft/ft.

The following stream cross-section is typical for the site area.



B. Solutions:

First we must determine the design frequency for the structure. Relating to the data on page 2 of the guide, we see that with a 75' bridge structure we are dealing with a Major Bridge and a design frequency of 50 years is required. A check on the 100 year frequency should sldo be made.

In terms of percent risk this means that a flood of 50 year recurrence interval has a 33% chance of occurring in a 20 year period, a 64% chance in 50 years and an 87% chance in 100 years (reference page 3).

Using this design frequency of 50 years, we can then determine the design flow by the various methods outlined in the guide.

a. Log-Pearson Type III Method

Since we have stream gage records available, analysis of this data should be the most reliable method. Normally this should be programmed on a computer for the efficient production.

Referring to the procedure outlined on pages 1 to 3, and the data under (a) above we have the following:

$\underline{Y}$	$\underline{X}$	$\underline{X^2}$	$\underline{X^3}$
2110	7.6544	58.5905	448.4777
3270	8.0925	65.4893	529.9750
1320	7.1854	51.6298	370.9800
2460	7.8079	60.9636	475.9984
2010	7.6059	57.8496	439.9974
1550	7.3460	53.9639	396.4191
1500	7.3132	53.4832	391.1344
1940	7.5704	57.3116	433.8743
2430	7.7956	60.7721	473.7578
1650	7.4085	54.8863	406.6270
2510	7.8280	61.2782	479.6879
3430	8.1403	66.2647	539.4159
1700	7.4384	55.3295	411.5624
3760	8.2322	67.7687	557.8836
3000	8.0064	64.1019	513.2235
	$\Sigma X=115.4251$	$\Sigma X^2=889.6829$	$\Sigma X^3=6869.0144$

Computing Mean:

$N = 15$ (No. of Events)

$\Sigma X = 115.4251$

$$M = \frac{\Sigma X}{N} = \frac{115.4251}{15} = 7.6950$$

$M = \underline{7.6950}$

Computing Standard deviation:

$\Sigma X^2 = 889.6829$

$\Sigma X = 115.4251$

$(\Sigma X)^2 = 13322.95371$

$N = 15$

$$S = \sqrt{\frac{\Sigma X^2 - (\Sigma X)^2/N}{N-1}}$$

$$= \sqrt{\frac{(889.6827) - (13322.95371/15}{(15-1)}}$$

$S = \underline{.3258}$

Compute coefficient of skewness:

$N = 15$

$N^2 = 225$

$\Sigma X^3 = 6869.0144$

$\Sigma X^2 = 889.6829$

$\Sigma X = 115.4251$

$(\Sigma X)^3 = 1537803.264$

$S = .3258$

$$g = \frac{N^2 \Sigma X^3 - 3N\Sigma X \Sigma X^2 + 2(\Sigma X)^3}{N(N-1)(N-2)S^3}$$

$$= \frac{(225)(6869.0144)-(3)(15)(115.4251)(889.6829)+2(1537803.264)}{(15)(14)(13)(.3258)}$$

$g = \underline{.0696}$

Compute logarithms of discharges:

$M = 7.6950$

$S = .3258$

$g = .0696$

From Table 2, for $g = +.0696$

Recurrence Interval	K Value
25 yr.	1.772
50 yr.	2.091
100 yr.	2.376

Then:

$\ln Q = M + KS$

$Q_{25} = \text{antiln } (7.6950 + (1.772)(.3258))$

$Q_{25} = \underline{3914 \text{ cfs}}$

$Q_{50} = \text{antiln } (7.6950 + (2.091)(.3258))$

$Q_{50} = \underline{4343 \text{ cfs}}$

Q_{100} = antlin (7.6950 + (2.376)(.3258))

Q_{100} = <u>4765 cfs</u>

Also:

Q_2 = 2189 cfs

Q_5 = 2889 cfs

Q_{10} = 3344 cfs

b. If stream gage records had not been available then use of the flood frequency regression equations would probably be the next most reliable method.

<u>From the data given we have</u>

A = 15.1 sq. miles

$S_t = 0$

<u>From page 8, Figure 2 we find that in this area</u>

P = 160

<u>From page 9, Figure 3 we find that in this area</u>

I = 5

Using these variables in the equations listed on page 11 (Appendix B) we have:

<u>25 yr. flood</u>

$$Q_{25} = 9.25(A)^{.85} (S_t+1)^{-.35} (P)^{.53}(I)^{.81}$$

$$Q_{25} = (9.25)(15.1)^{.85} (1)^{-.35} (160)^{.53} (5)^{.81}$$

= <u>5042 cfs</u>

<u>50 yr. flood</u>

$$Q_{50} = 14(A)^{.75} (S_t+1)^{-.20} (P)^{.76}$$

$$= (14)(15.1)^{.75} (1)^{-.20} (160)^{.76}$$

= <u>5076 cfs</u>

Also:

Q_2 = 2299 cfs

Q_5 = 3112 cfs

Q_{10} = 3585 cfs

Q_{100} = No Prediction

c. This next method gives only generalized flows and should only be used as a last resort or for general approximations only.

From the data given we have that

A = 15.1 sq. miles

Using this in the Figure on page 10, yield the following general data

Q_{10} = 2200 cfs

Q_{25} = 2800 cfs (interpolated)

Q_{50} = 3000 cfs

Also the maximum flood rate recorded in S.E. Alaska would provide a flow in this drainage of

Q_{max} = <u>8000 cfs</u>

d. The next method should always be used as a check on any of the other methods and considered in the final analysis.

From the data and stream cross-section we have the following:

<u>Given</u>: The following information determined from the field survey.

1. From observed high-water marks, cross-sectional area of flood flow = 301 sq. ft.

2. Wetted perimeter = 59 ft.

3. Slope of stream bed = 0.005 ft./ft. = hydraulic gradient for stable reach.

4. Channel roughness coefficient = 0.045 (Obtained by correlation of 4" gravels in Table 1-5, page 14.)

Required: Peak discharge high-water marks.

1. Determine hydraulic radius

$$R = \frac{\text{Area}}{\text{Wetted Perimeter}} = \frac{301}{59} = \underline{5.1}$$

2. $$\text{Velocity} = \frac{1.486R^{2/3}S^{1/2}}{n} = \frac{1.486}{.045}(5.1)^{2/3}(.005)^{1/2} =$$

$$\underline{6.93 \text{ ft./sec.}}$$

3. Discharge = AV = 6.93(301) = <u>2085 cfs</u>

For ease of computation, a nomograph for solving the Manning Equation is presented in Figure 5, page 13.

The discharge under 3 would probably be the last 2 to 5 year flood flow, thus

$$Q_5 = \underline{2085 \text{ cfs}}$$

C. <u>Conclusions</u>

For this design since gage station data was given, a design flow of 4300 cfs will be used and a check on a flow of 4800 cfs will be made.

Hydraulic Charts for the Selection of Highway Culverts

Hydraulic Engineering Circular No. 5 December 1965*

Prepared by the Hydraulics Branch, Bridge Division, Office of Engineering and Operations, Bureau of Public Roads, Washington, D. C. 20235

CONTENTS

*See note back of cover for revisions.

U. S. DEPARTMENT OF COMMERCE Bureau of Public Roads

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Circular No. 5 - Hydraulic Charts for the Selection of Highway Culverts (this Circular)

Circular No. 6 - Design of Roadside Drainage Channels (Discontinued. Available as Hydraulic Design Series No. 4.)

Circular No. 7 - A FORTRAN Program for the Hydraulic Design of Circular Culverts (Discontinued. Available as HY-1.)

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NOTE: This edition of Circular No. 5 contains only minor revisions of the previous edition of April 1964. Reference to Table 1 on page 5-11 is changed to Table 2; Chart 2, page 5-22 is revised slightly to reflect research results; and an error in pipe sizes is corrected on page 5-43.

U. S. DEPARTMENT OF COMMERCE

Bureau of Public Roads

HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS

Prepared by Lester A. Herr
Chief, Hydraulics Branch, Bridge Division

In Collaboration with Herbert G. Bossy
Highway Research Engineer, Hydraulic Research Division

Introduction

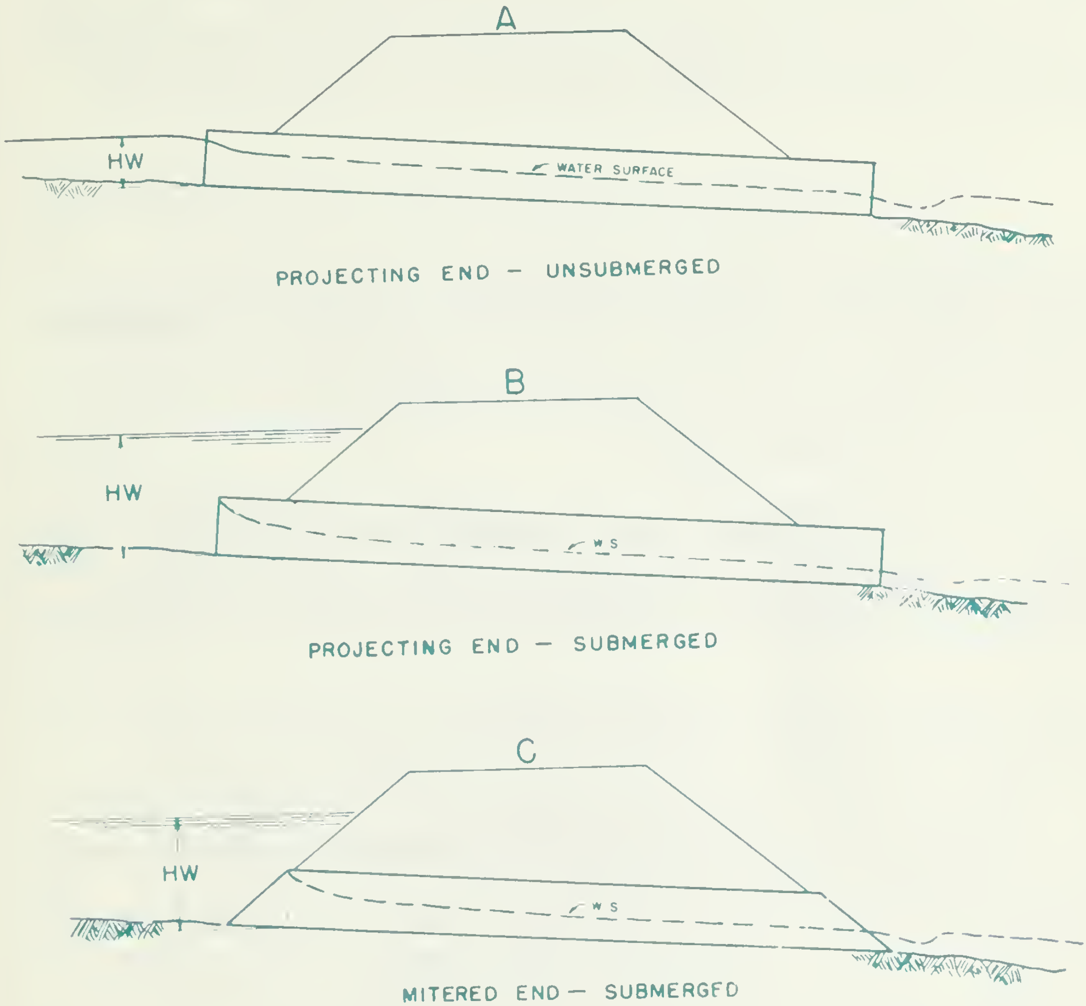
Designing highway culverts involves many factors including estimating flood peaks, hydraulic performance, structural adequacy, and overall construction and maintenance costs. This circular contains a brief discussion of the hydraulics of conventional culverts and charts for selecting a culvert size for a given set of conditions. Instructions for using the charts are provided. No attempt is made to cover all phases of culvert design. Subsequent circulars will cover culverts with modified inlets and outlets designed to increase performance or to apply to a particular location. Some approximations are made in the hydraulic design procedure for simplicity. These approximations are discussed at appropriate points throughout the circular.

For this discussion, conventional culverts include those commonly installed, such as circular, arch and oval pipes, both metal and concrete, and concrete box culverts. All such conventional culverts have a uniform barrel cross section throughout. The culvert inlet may consist of the culvert barrel projected from the roadway fill or mitered to the embankment slope. Sometimes inlets have headwalls, wingwalls and apron slabs, or standard end sections of concrete or metal. The more common types of conventional culverts are considered in this circular.

Culvert Hydraulics

Laboratory tests and field observations show two major types of culvert flow: (1) flow with inlet control and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness and length of the culvert barrel.

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a



INLET CONTROL

Figure 1

given set of conditions. The need for making these computations may be avoided, however, by computing headwater depths from the charts in this circular for both inlet control and outlet control and then using the higher value to indicate the type of control and to determine the headwater depth. This method of determining the type of control is accurate except for a few cases where the headwater is approximately the same for both types of control.

Both inlet control and outlet control types of flow are discussed briefly in the following paragraphs and procedures for the use of the charts are given.

Culverts Flowing With Inlet Control

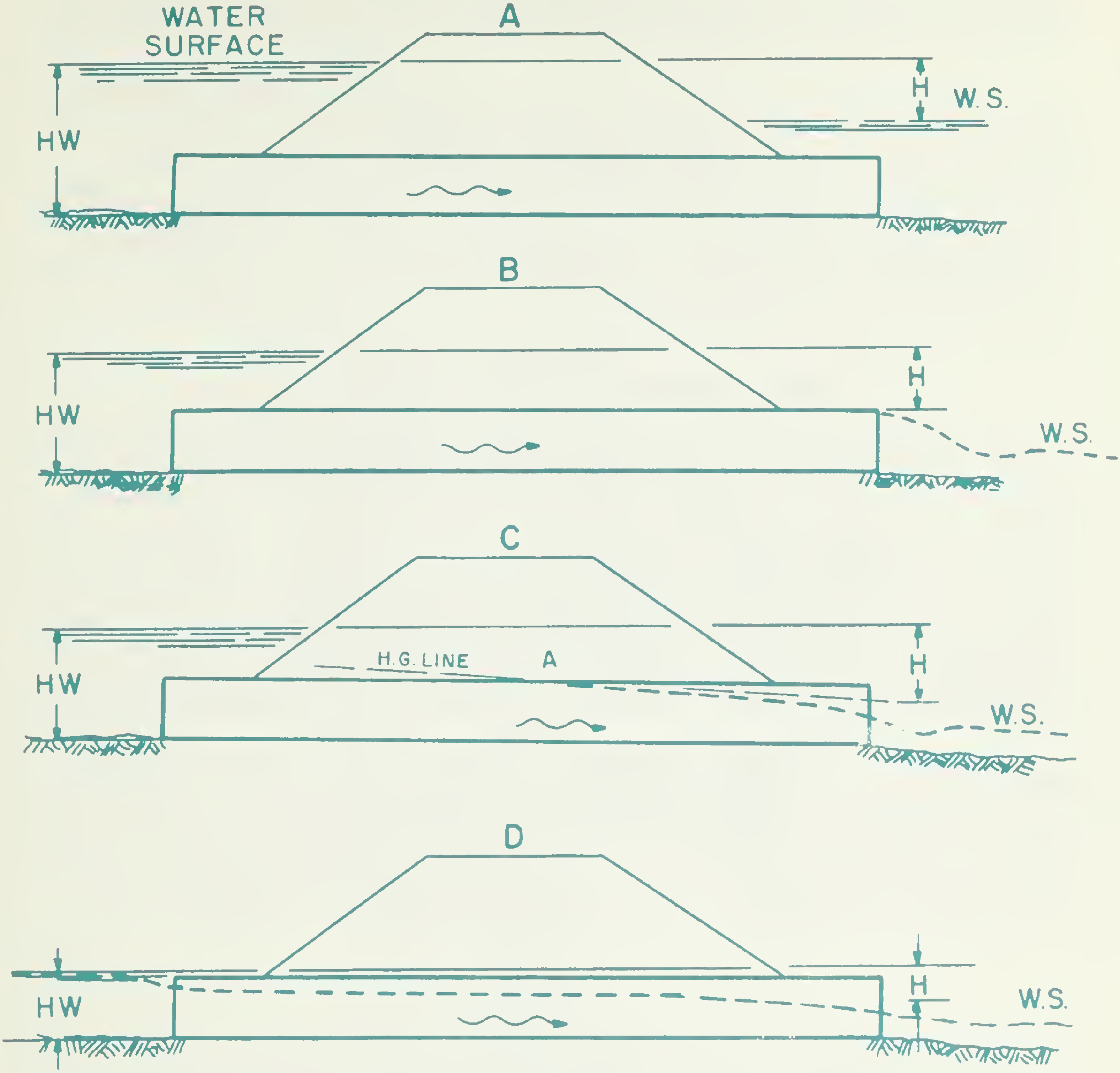
Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the entrance geometry, including the barrel shape and cross-sectional area, and the type of inlet edge. Sketches of inlet-control flow for both unsubmerged and submerged projecting entrances are shown in figures 1A and 1B. Figure 1C shows a mitered entrance flowing under a submerged condition with inlet control.

In inlet control the roughness and length of the culvert barrel and outlet conditions (including depth of tailwater) are not factors in determining culvert capacity. An increase in barrel slope reduces headwater to a small degree and any correction for slope can be neglected for conventional or commonly used culverts flowing with inlet control.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth (or headwater HW) is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth + velocity head). Because of the low velocities in most entrance pools and the difficulty in determining the velocity head for all flows, the water surface and the energy line at the entrance are assumed to be coincident, thus the headwater depths given by the inlet control charts in this circular can be higher than will occur in some installations. For the purposes of measuring headwater, the culvert invert at the entrance is the low point in the culvert opening at the beginning of the full cross-section of the culvert barrel.

Headwater-discharge relationships for the various types of circular and pipe-arch culverts flowing with inlet control are based on laboratory research with models and verified in some instances by prototype tests. This research is reported in National Bureau of Standards Report No. 4444[1/] entitled "Hydraulic Characteristics of Commonly Used Pipe Entrances", by John L. French and "Hydraulics of Conventional

1/ Available on loan from Division of Hydraulic Research, Bureau of Public Roads.



OUTLET CONTROL

Figure 2

Highway Culverts", by H. G. Bossy[2/]. Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the U. S. Geological Survey.

These research data were analyzed and nomographs for determining culvert capacity for inlet control were developed by the Division of Hydraulic Research, Bureau of Public Roads. These nomographs, Charts 1 through 6, give headwater-discharge relationships for most conventional culverts flowing with inlet control through a range of headwater depths and discharges. Chart No. 7, discussed on p. 5-13, is included in this revised edition to stress the importance of improving the inlets of culverts flowing with inlet control.

Culverts Flowing With Outlet Control

Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it, (see fig. 2). If the entire cross section of the barrel is filled with water for the total length of the barrel, the culvert is said to be in full flow or flowing full, figures 2A and 2B. Two other common types of outlet-control flow are shown in figures 2C and 2D. The procedures given in this circular provide methods for the accurate determination of headwater depth for the flow conditions shown in figures 2A, 2B and 2C. The method given for the part full flow condition, fig. 2D, gives a solution for headwater depth that decreases in accuracy as the headwater decreases.

The head H (fig. 2A) or energy required to pass a given quantity of water through a culvert flowing in outlet control with the barrel flowing full throughout its length is made up of three major parts. These three parts are usually expressed in feet of water and include a velocity head H_v, an entrance loss H_e, and a friction loss H_f. This energy is obtained from ponding of water at the entrance and expressed in equation form

$$H = H_v + H_e + H_f \qquad (1)$$

The velocity head H_v equals $\frac{V^2}{2g}$, where V is the mean or average velocity in the culvert barrel. (The mean velocity is the discharge Q, in cfs, divided by the cross-sectional area A, in sq. ft., of the barrel.)

The entrance loss H_e depends upon the geometry of the inlet edge. This loss is expressed as a coefficient k_e times the barrel velocity head or $H_e = k_e \frac{V^2}{2g}$. The entrance loss coefficients k_e for various types of entrances when the flow is in outlet control are given in Appendix B, Table 1, (p. 5-49).

2/ Presented at the Tenth National Conference, Hydraulics Division, A.S.C.E., August 1961. Available on loan from Division of Hydraulic Research, Bureau of Public Roads.

The friction loss H_f is the energy required to overcome the roughness of the culvert barrel. H_f can be expressed in several ways. Since most highway engineers are familiar with Manning's n the following expression is used:

$$H_f = \left[\frac{29 n^2 L}{R^{1.33}} \right] \frac{V^2}{2g}$$

where

- n = Manning's friction factor (see nomographs and page 5-30 for values)
- L = length of culvert barrel (ft.)
- V = mean velocity of flow in culvert barrel (ft./sec.)
- g = acceleration of gravity, 32.2 (ft./sec.^{2})
- R = hydraulic radius or $\frac{A}{WP}$ (ft.)

where

- A = area of flow for full cross-section (sq. ft.)
- WP = wetted perimeter (ft.)

Substituting in equation 1 and simplifying, we get for full flow

$$H = \left[1 + k_e + \frac{29 n^2 L}{R^{1.33}} \right] \frac{V^2}{2g} \qquad (2)$$

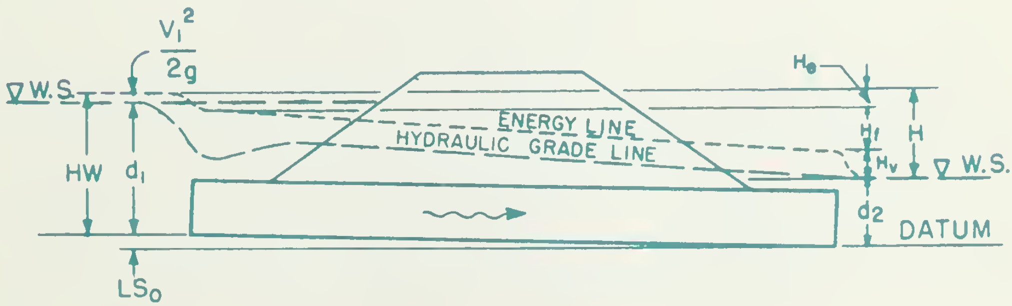


Figure 3

Figure 3 shows the terms of equation 2, the energy line, the hydraulic grade line and the headwater depth, HW. The energy line represents the total energy at any point along the culvert barrel. The hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length. The energy line and the pressure line are parallel over the length of the barrel except in the immediate vicinity of the inlet where the flow contracts and re-expands. The difference in elevation between these two lines is the velocity head, $\frac{V^2}{2g}$.

The expression for H is derived by equating the total energy upstream from the culvert entrance to the energy just inside the culvert outlet with consideration of all the major losses in energy. By referring to figure 3 and using the culvert invert at the outlet as a datum, we get:

$$d_1 + \frac{V_1^2}{2g} + LS_o = d_2 + H_v + H_e + H_f$$

where

d_1 and d_2 = depths of flow as shown in fig. 3

$\frac{V_1^2}{2g}$ = velocity head in entrance pool

LS_o = length of culvert times barrel slope

then

$$d_1 + \frac{V_1^2}{2g} + LS_o - d_2 = H_v + H_e + H_f$$

and

$$H = d_1 + \frac{V_1^2}{2g} + LS_o - d_2 = H_v + H_e + H_f$$

From the development of this energy equation and figure 3, head H is the difference between the elevations of the hydraulic grade line at the outlet and the energy line at the inlet. Since the velocity head in the entrance pool is usually small under ponded conditions, the water surface or headwater pool elevation can be assumed to equal the elevation of the energy line. Thus headwater elevations and headwater depths, as computed by the methods given in this circular, for outlet control, can be higher than might occur in some installations. Headwater depth is the vertical distance from the culvert invert at the entrance to the water surface, assuming the water surface (hydraulic grade line) and the energy line to be coincident, $d_1 + \frac{V_1^2}{2g}$ in figure 3.

Equation 2 can be solved for H readily by the use of the full-flow nomographs, Charts 8 through 14. Each nomograph is drawn for a particular barrel shape and material and a single value of n as noted on the respective charts. These nomographs can be used for other values of n by modifying the culvert length as directed in the instructions (p. 5-29) for the use of the full-flow nomographs.

In culvert design the depth of headwater HW or the elevation of the ponded water surface is usually desired. Finding the value of H from the nomographs or by equation 2 is only part of the solution for this headwater depth or elevation. In the case of figure 2A or figure 3, where the outlet is totally submerged, the headwater pool elevation (assumed to be the same elevation as the energy line) is found by adding H to the elevation of the tailwater. The headwater depth is the difference in elevations of the pool surface and the culvert invert at the entrance.

When the tailwater is below the crown of the culvert, the submerged condition discussed above no longer exists and the determination of headwater is somewhat more difficult. In discussing outlet-control flow for this condition, tailwater will be assumed to be so low that it has no effect on the culvert flow. (The effect of tailwater will be discussed later.) The common types of flow for the low tailwater condition are shown in figures 2B, 2C and 2D. Each of these flow conditions are dependent on the amount of discharge and the shape of the culvert cross section. Each condition will be discussed separately.

Full flow at the outlet, figure 2B, will occur only with the higher rates of discharge. Charts 15 through 20 are provided to aid in determining this full flow condition. The curves shown on these charts give the depth of flow at the outlet for a given discharge when a culvert is flowing with outlet control. This depth is called critical depth d_c. When the discharge is sufficient to give a critical depth equal to the crown of the culvert barrel, full flow exists at the outlet as in figure 2B. The hydraulic grade line will pass through the crown of the culvert at the outlet for all discharges greater than the discharge causing critical depth to reach the crown of the culvert. Head H can be measured from the crown of the culvert in computing the water surface elevation of the headwater pool.

When critical depth falls below the crown of the culvert at the outlet, the water surface drops as shown in either figures 2C or 2D, depending again on the discharge. To accurately determine headwater for these conditions, computations for locating a backwater curve are usually required. These backwater computations are tedious and time consuming and they should be avoided if possible. Fortunately, headwater for the flow condition shown in figure 2C can be solved by using the nomographs and the instructions given in this circular.

For the condition shown in figure 2C, the culvert must flow full for part of its length. The hydraulic grade line for the portion of the length in full flow will pass through a point where the water breaks with the top of the culvert as represented by point A in figure 2C. Backwater computations show that the hydraulic grade line if extended as a straight line will cut the plane of the outlet cross section at a point above critical depth (water surface). This point is at a height approximately equal to one half the distance between critical depth and the crown of the culvert. The elevation of this point can be used as an equivalent hydraulic grade line and H, as determined by equation 2 or the nomographs, can be added to this elevation to find the water surface elevation of the headwater pool.

The full flow condition for part of the barrel length, figure 2C, will exist when the headwater depth HW, as computed from the above headwater pool elevation, is equal to or greater than the quantity

$$D + (1 + k_e) \frac{V^2}{2g}$$

where V is the mean velocity for the full cross section of the barrel; k_e, the entrance loss coefficient; and D, the inside height of the culvert. If the headwater is less than the above value, a free water surface, figure 2D, will extend through the culvert barrel.

The part full flow condition of figure 2D must be solved by a backwater computation if accurate headwater depths are desired. Details for making this computation are not given in this circular. Instead the solution used is the same as that given for the flow condition of figure 2C, with the reservation that headwater depths become less accurate as the discharge for a particular culvert decreases. Generally, for design purposes, this method is satisfactory for head water depths above 0.75D, where D is the height of the culvert barrel. Culvert capacity charts found in Hydraulic Engineering Circular No. 10 give a more accurate and easy solution for this free surface flow condition.

Headwater depth HW can be expressed by a common equation for all outlet-control conditions, including all depths of tailwater. This is accomplished by designating the vertical dimension from the culvert invert at the outlet to the elevation from which H is measured as h_o. The headwater depth HW equation is

$$HW = H + h_o - LS_o \tag{3}$$

All the terms in this equation are in feet. H is computed by equation 2 or found from the full-flow nomographs. L is the length of culvert in feet and S_o the barrel slope in ft. per ft. The distance h_o is discussed in the following paragraphs for the various conditions of outlet-control flow. Headwater HW is the distance in feet from the invert of the culvert at the inlet to the water surface of the headwater pool.

When the elevation of the water surface in the outlet channel is equal to or above the elevation of the top of the culvert opening at the outlet, figure 2A, h_o is equal to the tailwater depth. Tailwater depth TW is the distance in feet from the culvert invert at the outlet to the water surface in the outlet channel. The relationship of HW to the other terms in equation 3 is illustrated in figure 4.

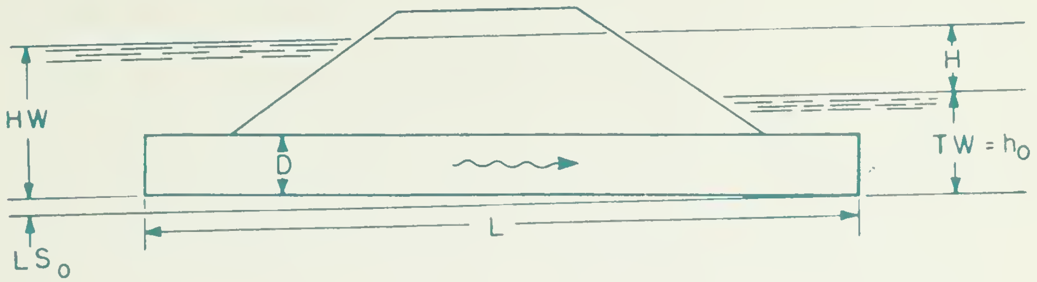


Figure 4

If the tailwater elevation is below the top of the culvert opening at the outlet, figure 2B, 2C and 2D, h_o is more difficult to determine. The discharge, size and shape of culvert, and the TW must be considered. In these cases, h_o is the greater of two values (1) TW depth as defined above or (2) $\frac{d_c + D}{2}$. The latter dimension is the distance to the equivalent hydraulic grade line discussed previously. In this fraction d_c is the critical depth, as read from Charts 15 through 20 and D is the culvert height. The value of d_c can never exceed D, making the upper limit of this fraction equal to D. Where TW is the greater of these two values, critical depth is submerged sufficiently to make TW effective in increasing the headwater. The sketch in figure 5 shows the terms of equation 3 for this low tailwater condition. Figure 5 is drawn similar to figure 2C, but a change in discharge can change the water surface profile to that of figure 2B or 2D.

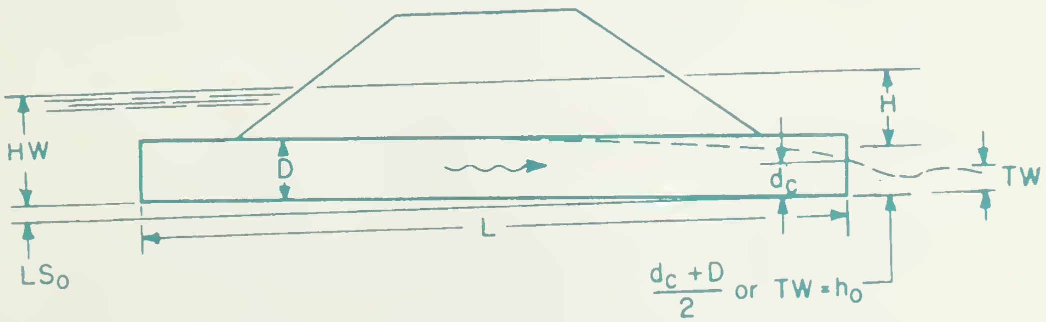


Figure 5

Computing Depth of Tailwater

In culverts flowing with outlet control, tailwater can be an important factor in computing both the headwater depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

Much engineering judgment and experience is needed to evaluate possible tailwater conditions during floods. A field inspection should be made to check on downstream controls and to determine water stages. Oftentimes tailwater is controlled by a downstream obstruction or by water stages in another stream. Fortunately, most natural channels are wide compared to the culvert and the depth of water in the natural channel is considerably less than critical depth, thus the tailwater is ineffective and channel depth computations are not always warranted.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation (see page 5-12) if the channel is reasonably uniform in cross section, slope and roughness. Values of n for natural streams for use in Manning's equation may be found in Table 2, appendix B, p. 5-50. If the water surface in the outlet channel is established by downstream controls, other means must be found to determine the tailwater elevation. Sometimes this necessitates a study of the stage-discharge relationship of another stream into which the stream in question flows or the securing of data on reservoir elevations if a storage dam is involved.

Velocity of Culvert Flow

A culvert, because of its hydraulic characteristics, increases the velocity of flow over that in the natural channel. High velocities are most damaging just downstream from the culvert outlet and the erosion potential at this point is a feature to be considered in culvert design.

Energy dissipators for channel flow have been investigated in the laboratory and many have been constructed, especially in irrigation channels. Designs for highway use have been developed and constructed at culvert outlets. All energy dissipators add to the cost of a culvert, therefore, they should be used only to prevent or to correct a serious erosion problem. (See reference 5, p. 5-14.)

The judgment of engineers working in a particular area is required to determine the need for energy dissipators at culvert outlets. As an aid in evaluating this need, culvert outlet velocities should be computed. These computed velocities can be compared with outlet velocities of alternate culvert designs, existing culverts in the area, or the natural stream velocities. In many streams the maximum velocity in the main channel is considerably higher than the mean velocity for the whole channel cross-section. Culvert outlet velocities should be compared with maximum stream velocities in determining the need for channel protection. A change in size of culvert does not change outlet velocities appreciably in most cases.

Outlet velocities for culverts flowing with inlet control may be approximated by computing the mean velocity for the culvert cross section using Manning's equation

$$V = \frac{1.49}{n} R^{2/3} \quad S_o^{1/2}$$

Since the depth of flow is not known the use of tables or charts is recommended in solving this equation[3/]. The outlet velocity as computed by this method will usually be high because the normal depth, assumed in using Manning's equation, is seldom reached in the relatively short length of the average culvert. Also, the shape of the outlet channel, including aprons and wingwalls, have much to do with changing the velocity occurring at the end of the culvert barrel. Tailwater is not considered effective in reducing outlet velocities for most inlet control conditions.

In outlet control, the average outlet velocity will be the discharge divided by the cross-sectional area of flow at the outlet. This flow area can be either that corresponding to critical depth, tailwater depth (if below the crown of the culvert) or the full cross section of the culvert barrel.

Performance Curves

Although the procedure given in this circular is primarily for use in selecting a size of culvert to pass a given discharge at a given headwater, a better understanding of culvert operation can be gained by plotting performance curves through some range of discharges and barrel slopes. Such curves can also be used to compare the performance of different sizes and types of culverts. The construction of such curves is described in Appendix A, page 5-45.

Inlets and Culvert Capacity

Inlet shape, edge geometry and skew of the entrance affects culvert capacity. Both the shape and edge geometry have been investigated by recent research but the effect of skew for various flow conditions has not been examined. Results show that the inlet edge geometry is particularly important to culvert performance in inlet-control flow. A comparison of several types of commonly used inlets can be made by referring to charts 2 and 5. The type of inlet has some effect on capacity in outlet control but generally the edge geometry is less important than in inlet control.

3/ See references page 5-14.

As shown by the inlet control nomograph on Chart 5, the capacity of a thin edge projecting metal pipe can be increased by incorporating the thin edge in a headwall. The capacity of the same thin edged pipe can be further increased if the entrance is rounded, bevelled or tapered by the addition of an attachment or the building of these shapes into a headwall. Although research on improving culvert entrances is not complete, sufficient data are available to permit the construction of Chart 7, an inlet control nomograph for the performance of a bevelled inlet on a circular culvert. A sketch on the nomograph shows the dimensions of two possible bevels. Although nomographs have not been prepared for other barrel shapes, the capacity of box culverts can be increased at little cost by incorporating a bevel into the headwall. In computing headwater depths for outlet control, when the above bevel is used, k_e equals 0.25 for corrugated metal barrels and 0.2 for concrete barrels.

Figure 6 shows a photograph of a bevel constructed in the headwall of a corrugated metal pipe.

Photo -- Courtesy of Oregon State Highway Department

Figure 6

REFERENCES

1. "Hydraulic Tables", Corps of Engineers, U. S. Army. For sale by Superintendent of Documents, Government Printing Office, Washington, D. C. Price $2.75.

2. "Hydraulic and Excavation Tables", U. S. Bureau of Reclamation. For sale by Superintendent of Documents, Government Printing Office, Washington, D. C. Price $1.50.

3. "Handbook of Hydraulics", by H. W. King, McGraw-Hill Book Company, New York City.

4. "Design Charts for Open-Channel Flow", U. S. Department of Commerce, Bureau of Public Roads. For sale by Superintendent of Documents, Government Printing Office, Washington, D. C. Price 70 cents.

5. "Hydraulic Design of Stilling Basins and Energy Dissipators", by A. J. Peterka, U. S. Department of Interior, Bureau of Reclamation, 1964. For sale by the Superintendent of Documents, Government Printing Office, Washington, D. C., 20402 or the Chief Engineer, Bureau of Reclamation, Attention 841, Denver Federal Center, Denver, Colorado, 80225. Price $1.75.

Procedure for Selection of Culvert Size

Step 1: List design data. (See suggested tabulation form, figure 7, p. 5-18.)

a. Design discharge Q, in cfs., with average return period. (i.e. Q_{25} or Q_{50} etc.)

b. Approximate length L of culvert, in feet.

c. Slope of culvert. (If grade is given in percent, convert to slope in ft. per ft.)

d. Allowable headwater depth, in feet, which is the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert.

e. Mean and maximum flood velocities in natural stream.

f. Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2: Determine the first trial size culvert.

Since the procedure given is one of trial and error, the initial trial size can be determined in several ways:

a. By arbitrary selection.

b. By using an approximating equation such as $\frac{Q}{10} = A$ from which the trial culvert dimensions are determined.

c. By using inlet control nomographs (Charts 1-7) for the culvert type selected. If this method is used an $\frac{HW}{D}$ must be assumed, say $\frac{HW}{D} = 1.5$, and using the given Q a trial size is determined.

If any trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge equally between the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should be considered. Final selection should be based on an economic analysis.

Step 3: Find headwater depth for trial size culvert.

a. Assuming INLET CONTROL

(1) Using the trial size from step 2, find the headwater depth HW by use of the appropriate inlet control nomograph (Charts 1-7). Tailwater TW conditions are to be neglected in this determination. HW in this case is found by multiplying $\frac{HW}{D}$ obtained from the nomographs by the height of culvert D.

(2) If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control before computing HW for outlet control.

b. Assuming OUTLET CONTROL

(1) Approximate the depth of tailwater TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel. (See general discussion on tailwater, p. 5-11.)

(2) For tailwater TW elevation <u>equal to or greater than</u> the top of the culvert at the outlet set h_o equal to TW and find HW by the following equation (equation 3).

$$HW = H + h_o - LS_o$$

where

- HW = vertical distance in feet from culvert invert (flow line) at entrance to the pool surface.
- H = head loss in feet as determined from the appropriate nomograph (Charts 8-14)
- h_o = vertical distance in feet from culvert invert at outlet to the hydraulic grade line (In this case h_o equals TW, measured in feet above the culvert invert.)
- S_o = slope of barrel in ft./ft.
- L = culvert length in ft.

(3) For tailwater TW elevations <u>less than</u> the top of the culvert at the outlet, find headwater HW by equation 3 as in b(2) above except that

$$h_o = \frac{d_c + D}{2} \text{ or TW, whichever is the greater.}$$

where

- d_c = critical depth in ft. (Charts 15 through 20) Note: d_c cannot exceed D
- D = height of culvert opening in ft.

Note: Headwater depth determined in b(3) becomes increasingly less accurate as the headwater computed by this method falls below the value $D + (1 + k_e)\frac{V^2}{2g}$. (See discussion under "Culvert Flowing Full with Outlet Control", p. 5-9.)

c. Compare the headwaters found in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

d. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 3b. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a.)

Step 4: Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

a. If outlet control governs in Step 3c above, outlet velocity equals $\frac{Q}{A_o}$, where A_o is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel use A_o corresponding to d_c or TW depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel.

b. If inlet control governs in step 3c, outlet velocity can be assumed to equal mean velocity in open-channel flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.

Note: Charts and tables are helpful in computing outlet velocities. (See references p. 5-14.)

Step 6: Record final selection of culvert with size, type, required headwater, outlet velocity, and economic justification.

PROJECT: ______________ DESIGNER: ______________

DATE: ______________

HYDROLOGIC AND CHANNEL INFORMATION

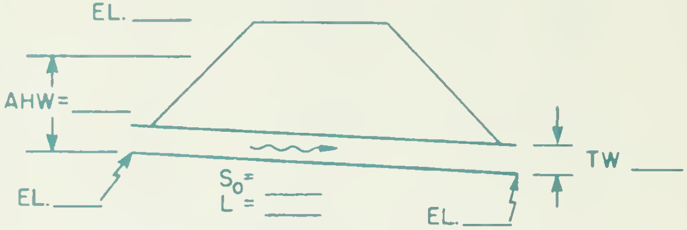
Q_1 = ______ TW_1 = ______

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ______________



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL, $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				

SUMMARY & RECOMMENDATIONS:

Figure 7

INLET-CONTROL NOMOGRAPHS

Charts 1 through 7

Instructions for Use

1. To determine headwater (HW), given Q, and size and type of culvert.

   a. Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or $\frac{Q}{B}$ for box culverts; mark intersection of straightedge on $\frac{HW}{D}$ scale marked (1).

   b. If $\frac{HW}{D}$ scale marked (1) represents entrance type used, read $\frac{HW}{D}$ on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read $\frac{HW}{D}$.

   c. Compute HW by multiplying $\frac{HW}{D}$ by D.

2. To determine discharge (Q) per barrel, given HW, and size and type of culvert.

   a. Compute $\frac{HW}{D}$ for given conditions.

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ scale (1) as found in (b) above and the size of culvert on the left scale. Read Q or $\frac{Q}{B}$ on the discharge scale.

   d. If $\frac{Q}{B}$ is read in (c) multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, allowable HW and type of culvert.

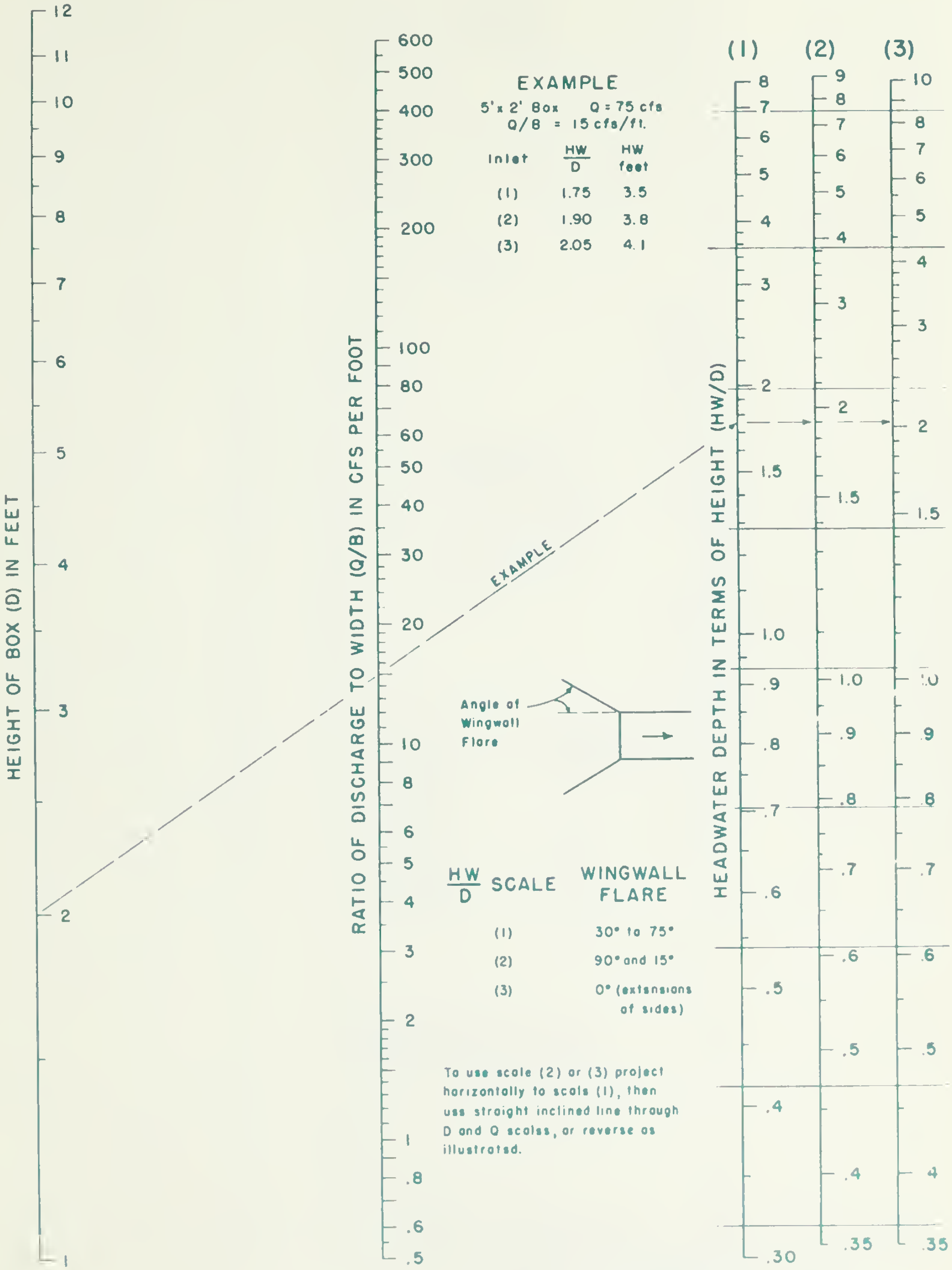
   a. Using a trial size, compute $\frac{HW}{D}$.

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ scale (1) as found in (b) above to given discharge and read diameter, height or size of culvert required for $\frac{HW}{D}$ value.

   d. If D is not that originally assumed, repeat procedure with a new D.

CHART I



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

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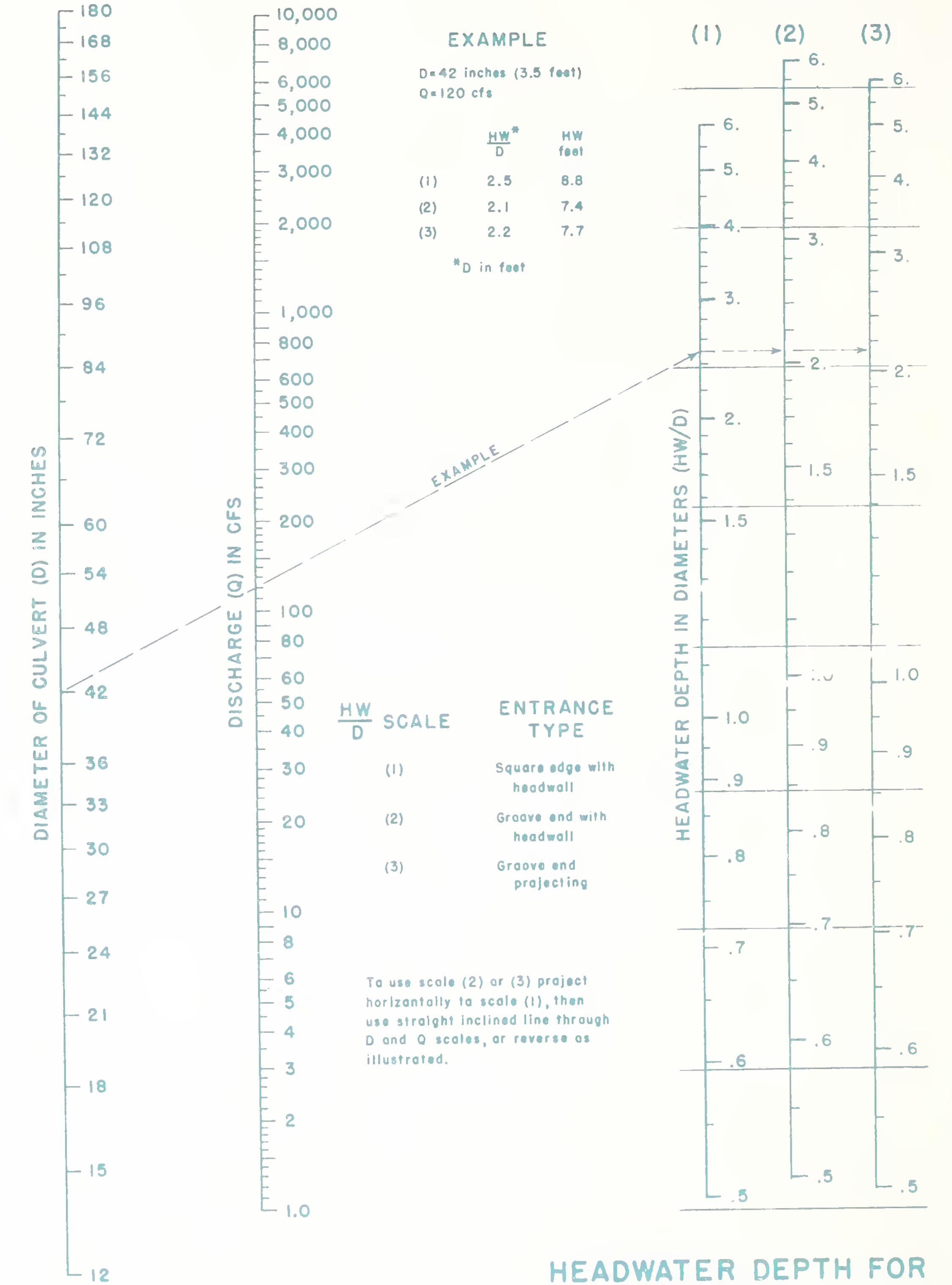
CHART 2



HEADWATER SCALES 2&3
REVISED MAY 1964

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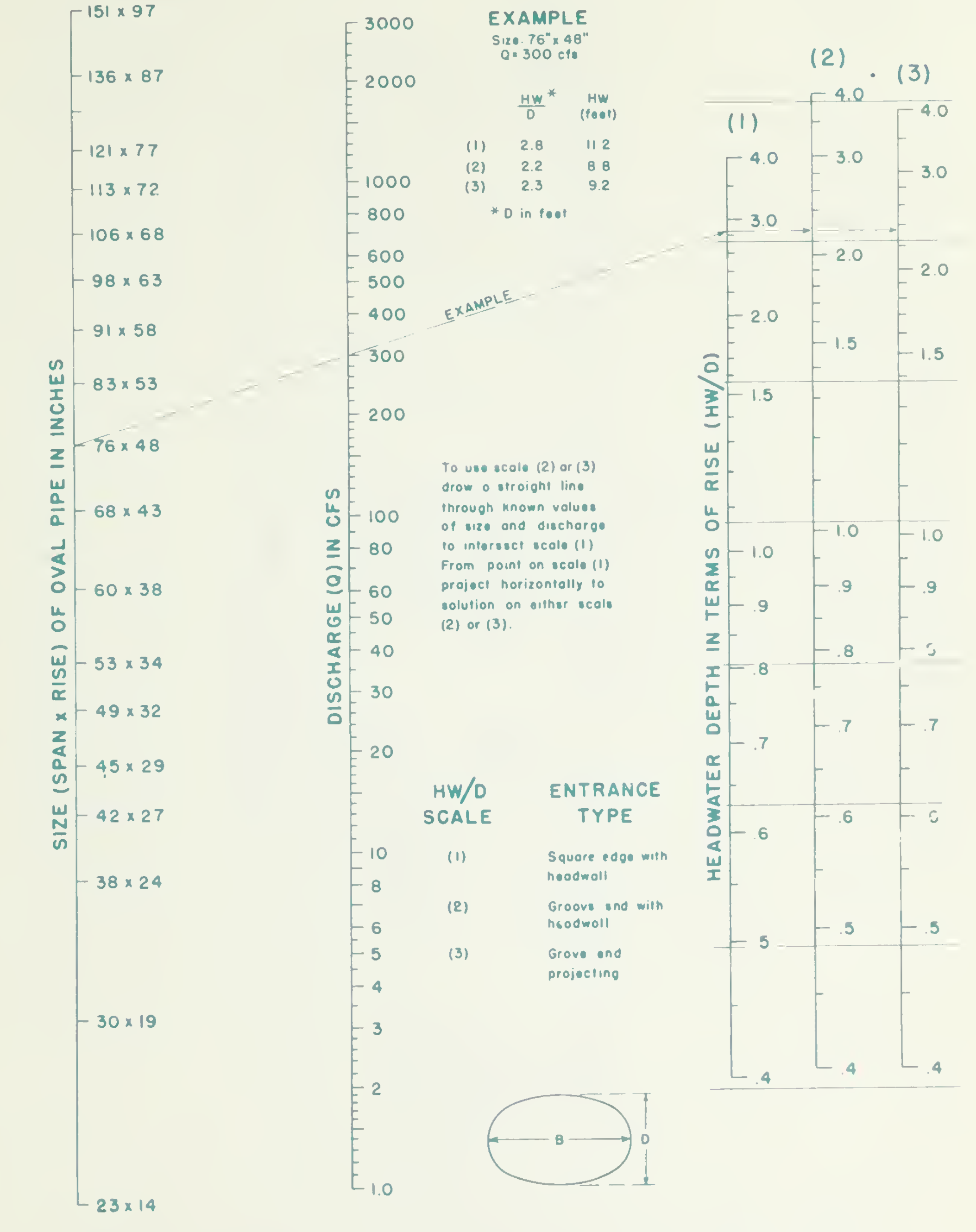
HEADWATER DEPTH FOR
CONCRETE PIPE CULVERTS
WITH INLET CONTROL



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

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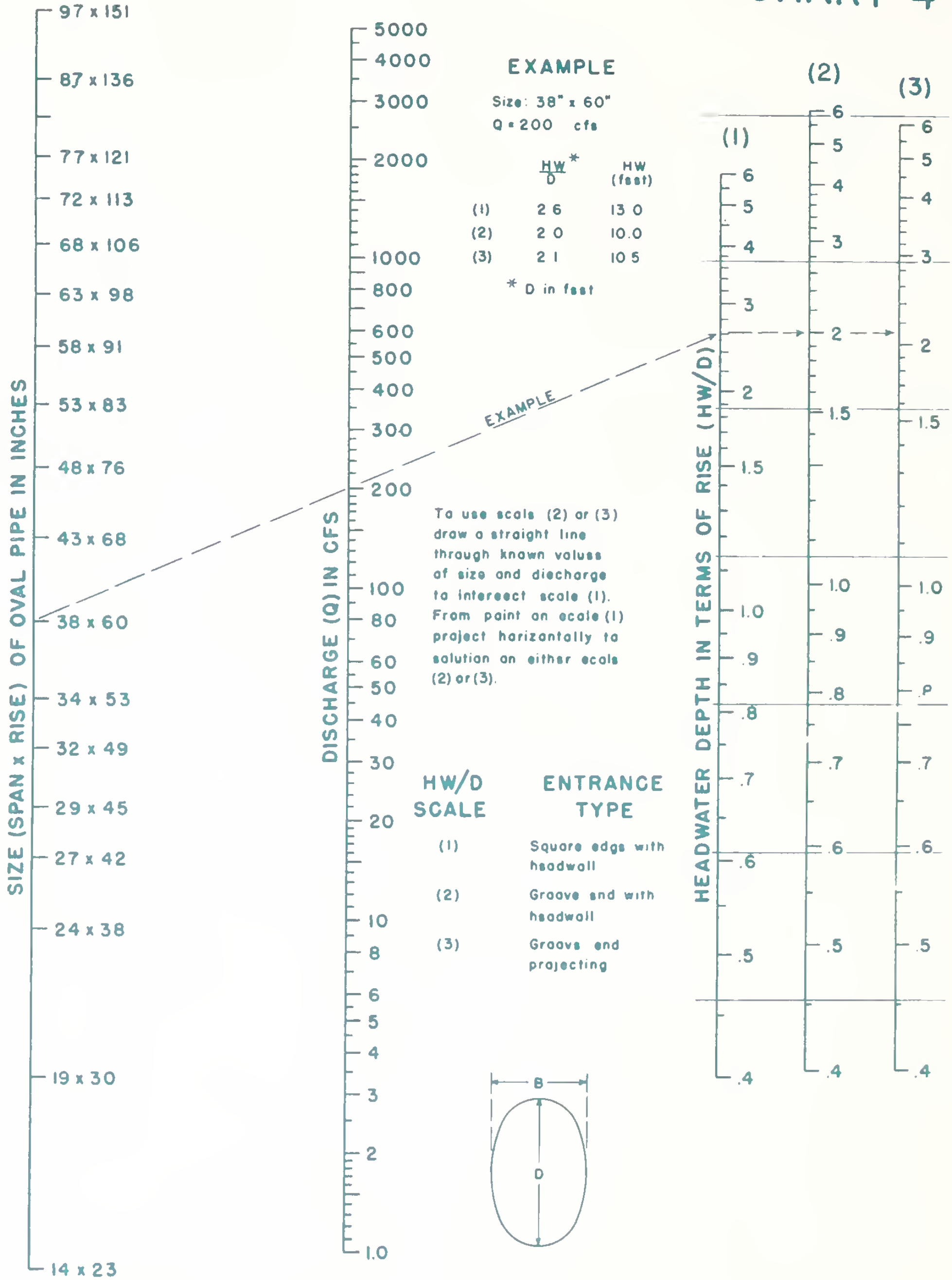
CHART 4



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

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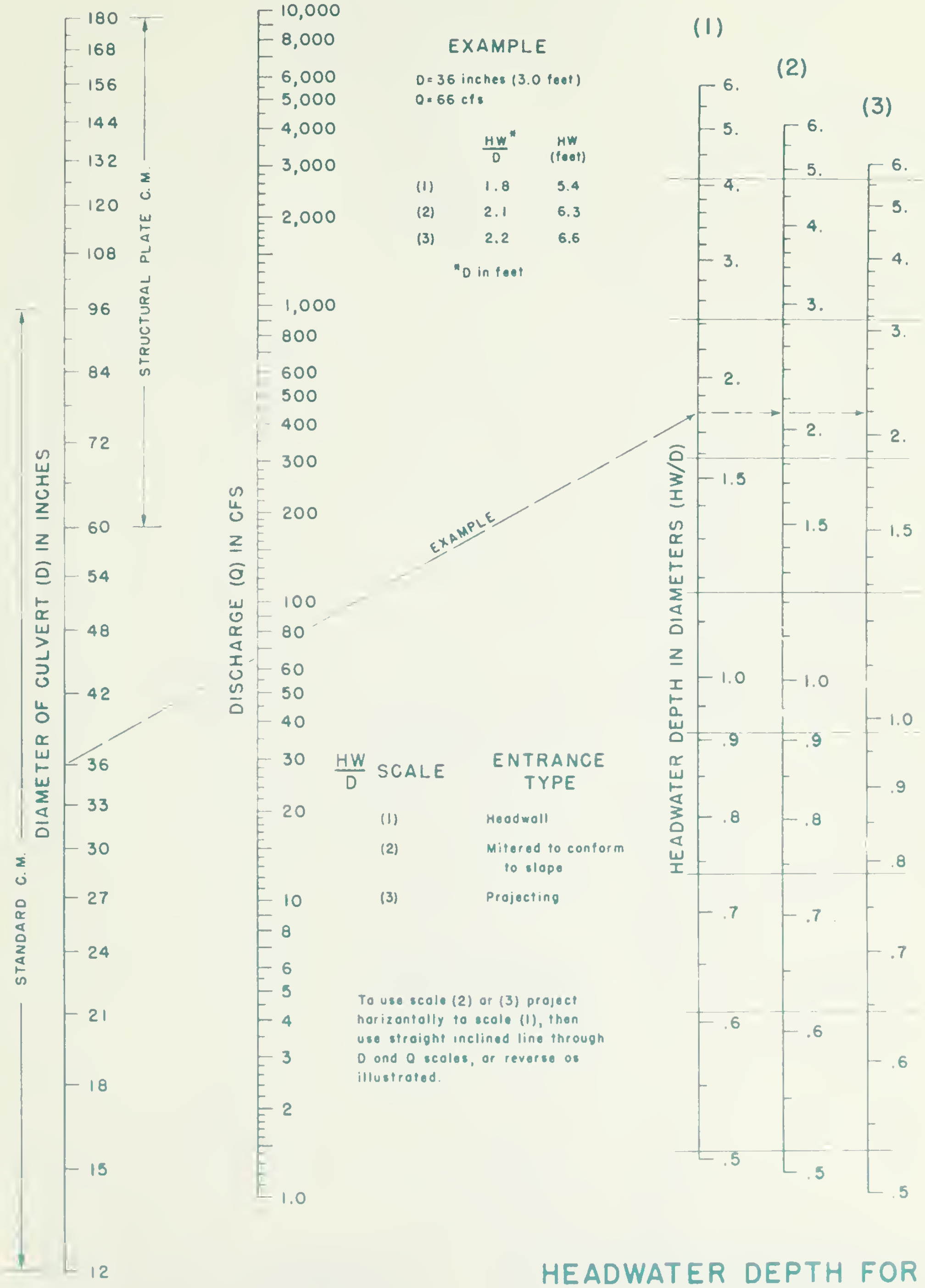
CHART 5



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

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* ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

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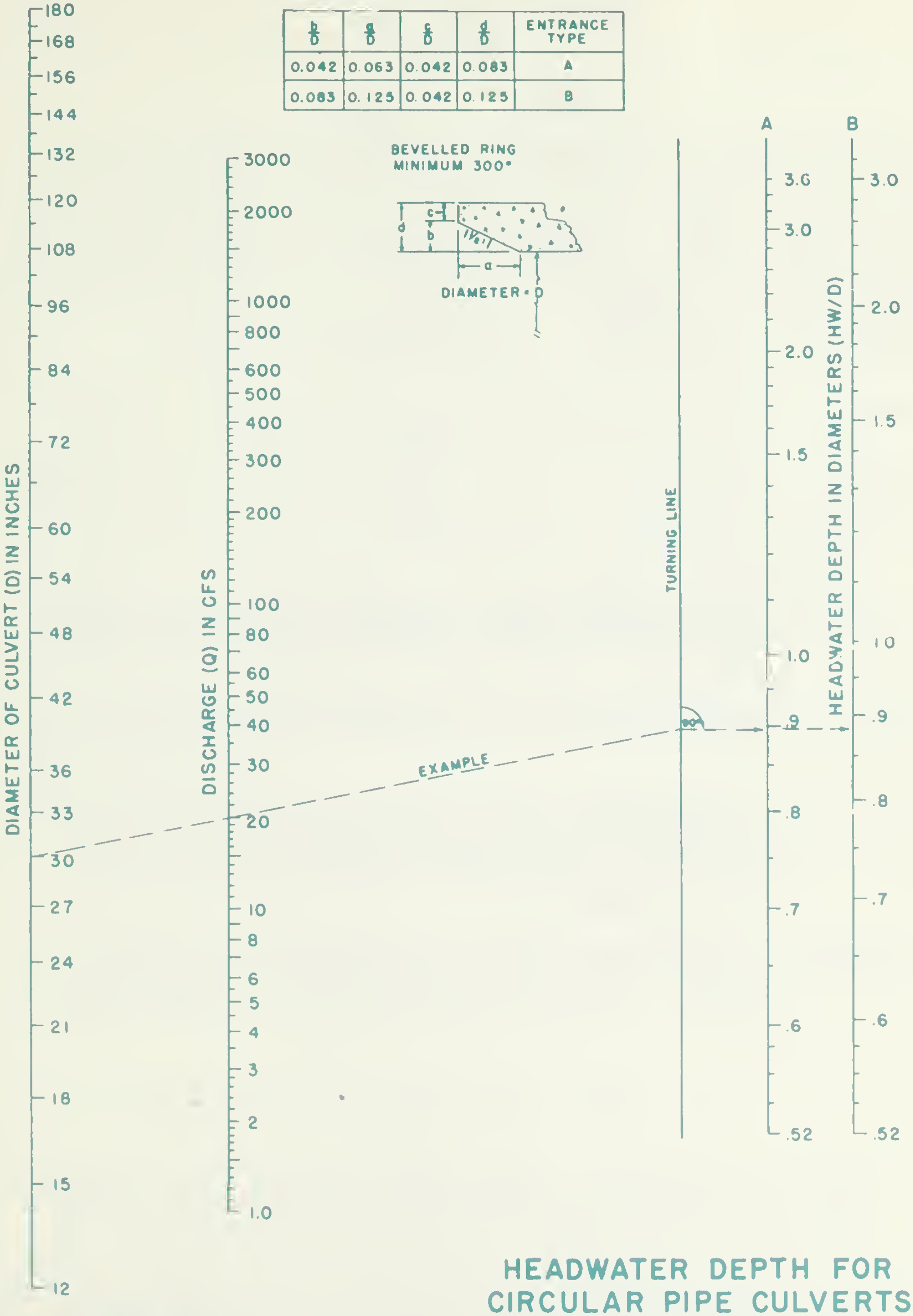
HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

CHART 7

b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



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HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELLED RING INLET CONTROL

OUTLET-CONTROL NOMOGRAPHS

Charts 8 through 14

Instructions for Use

Outlet control nomographs solve equation 2, p. 5-6, for head H when the culvert barrel flows full for its entire length. They are also used to determine head H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW, since they only give H in equation 3, $HW = H + h_o - LS_o$. (See discussion for "Culverts Flowing with Outlet Control", p. 5-5.)

1. To determine head H for a given culvert and discharge Q.

   a. Locate appropriate nomograph for type of culvert selected. Find k_e for entrance type in Appendix B, Table 1, p. 5-49.

   b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales follow instructions below:

      (1) If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper k_e and locate the starting point at the given culvert length. If a k_e curve is not shown for the selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.

      (2) For the n of the nomograph and a k_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the k_e values.

      (3) For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula

      $$L_1 = L \left[\frac{n_1}{n}\right]^2$$ See instruction 2 for n values.

   c. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line". See instruction 3 below for size considerations for rectangular box culvert.

   d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving equation 2, p. 5-6.

2. Values of n for commonly used culvert materials.

Concrete

Pipe	Boxes
0.012	0.012

Corrugated Metal

	Small Corrugations (2 2/3" x 1/2")	Medium Corrugations (3" x 1")	Large Corrugations (6" x 2")
Unpaved	0.024	0.027	Varies*
25% paved	0.021	0.023	0.026
Fully paved	0.012	0.012	0.012

*Variation in n with diameter shown on charts. The various n values have been incorporated into the nomographs and no adjustment for culvert length is required as instructed in 1b(3).

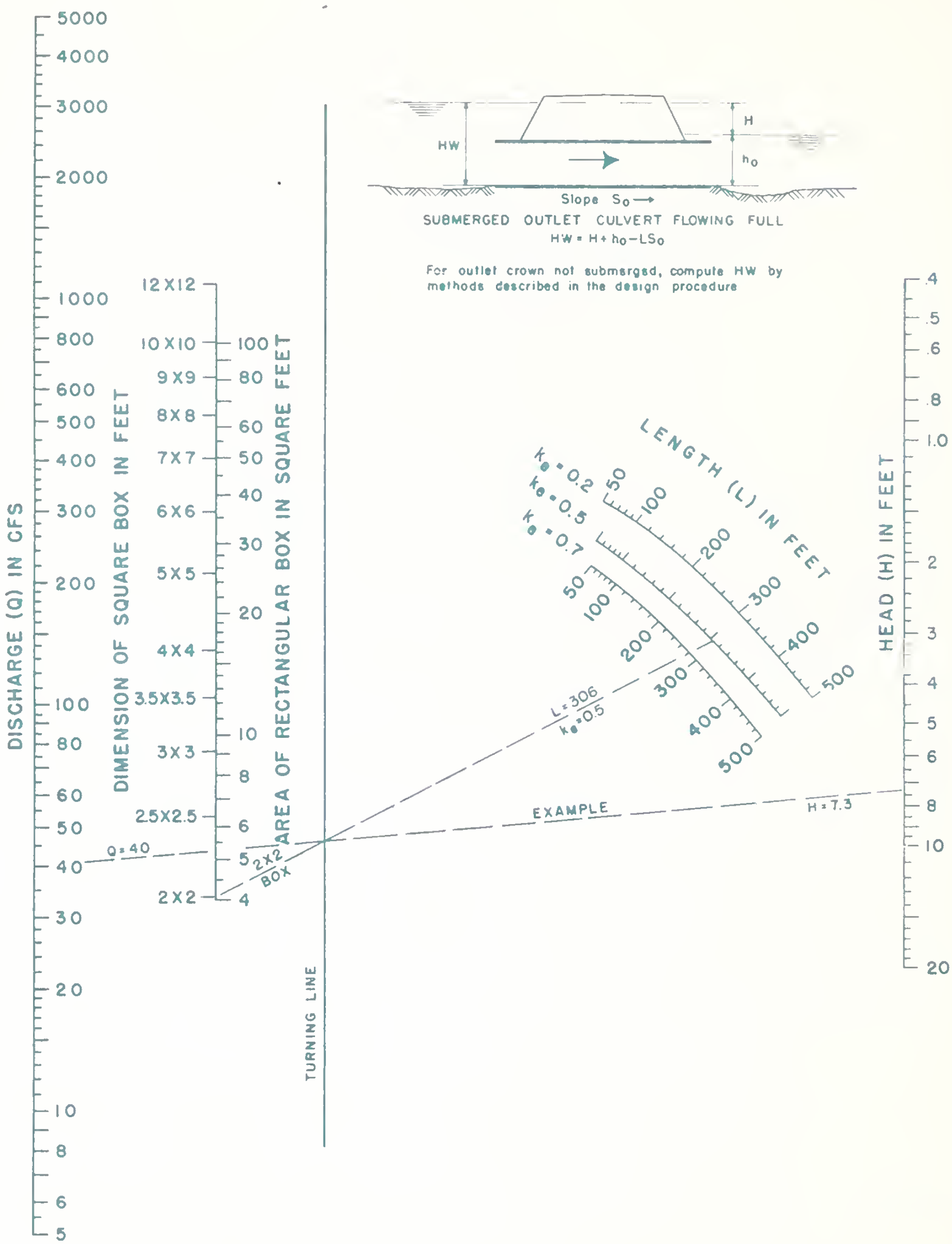
3. To use the box culvert nomograph, chart 8, for full-flow for other than square boxes.

a. Compute cross-sectional area of the rectangular box.

b. Connect proper point (see instruction 1) on length scale to barrel area[4/] and mark point on turning line.

c. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale.

4/ The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions use equation 2 for more accurate results.



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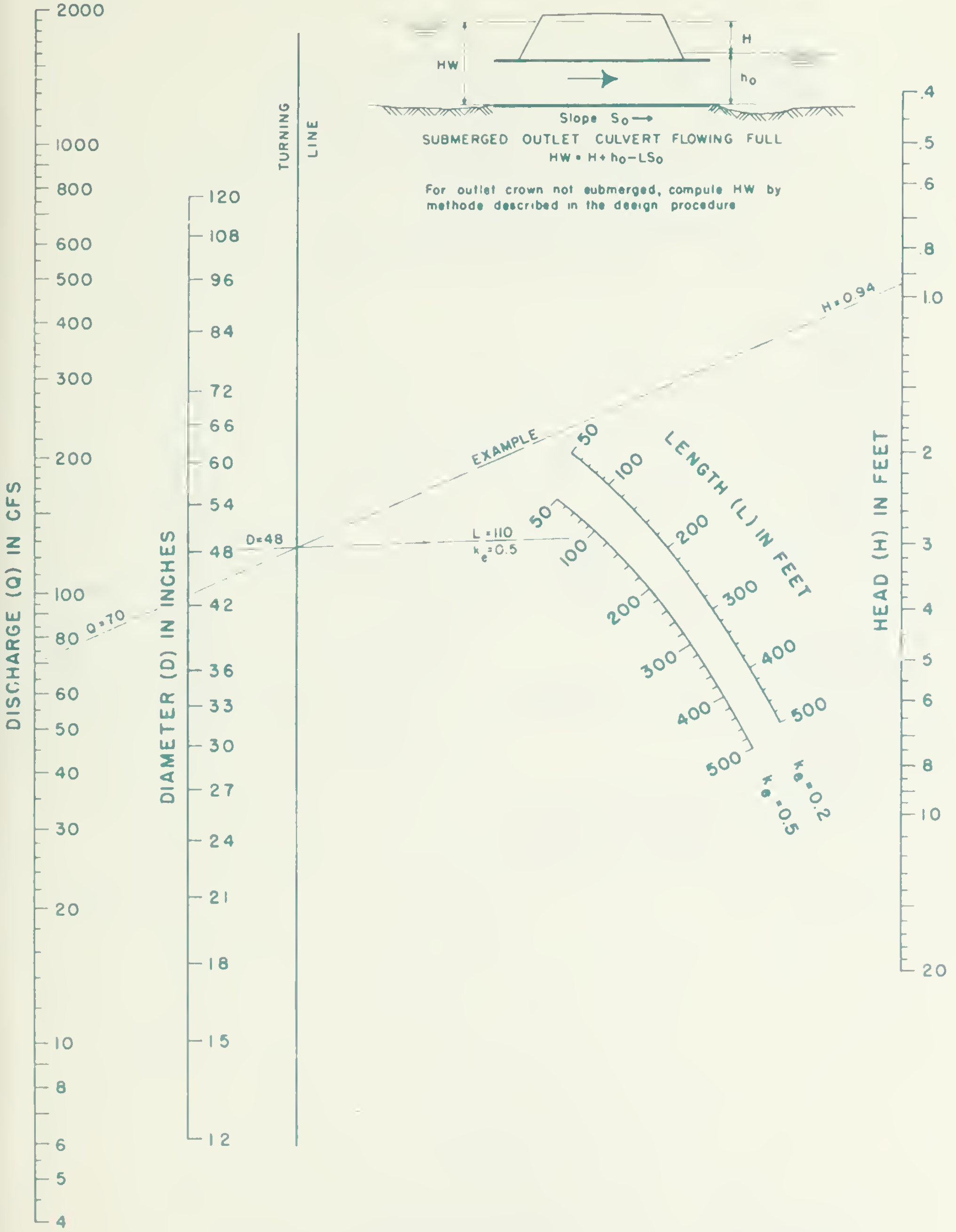
HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL

$n = 0.012$



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
$n = 0.012$

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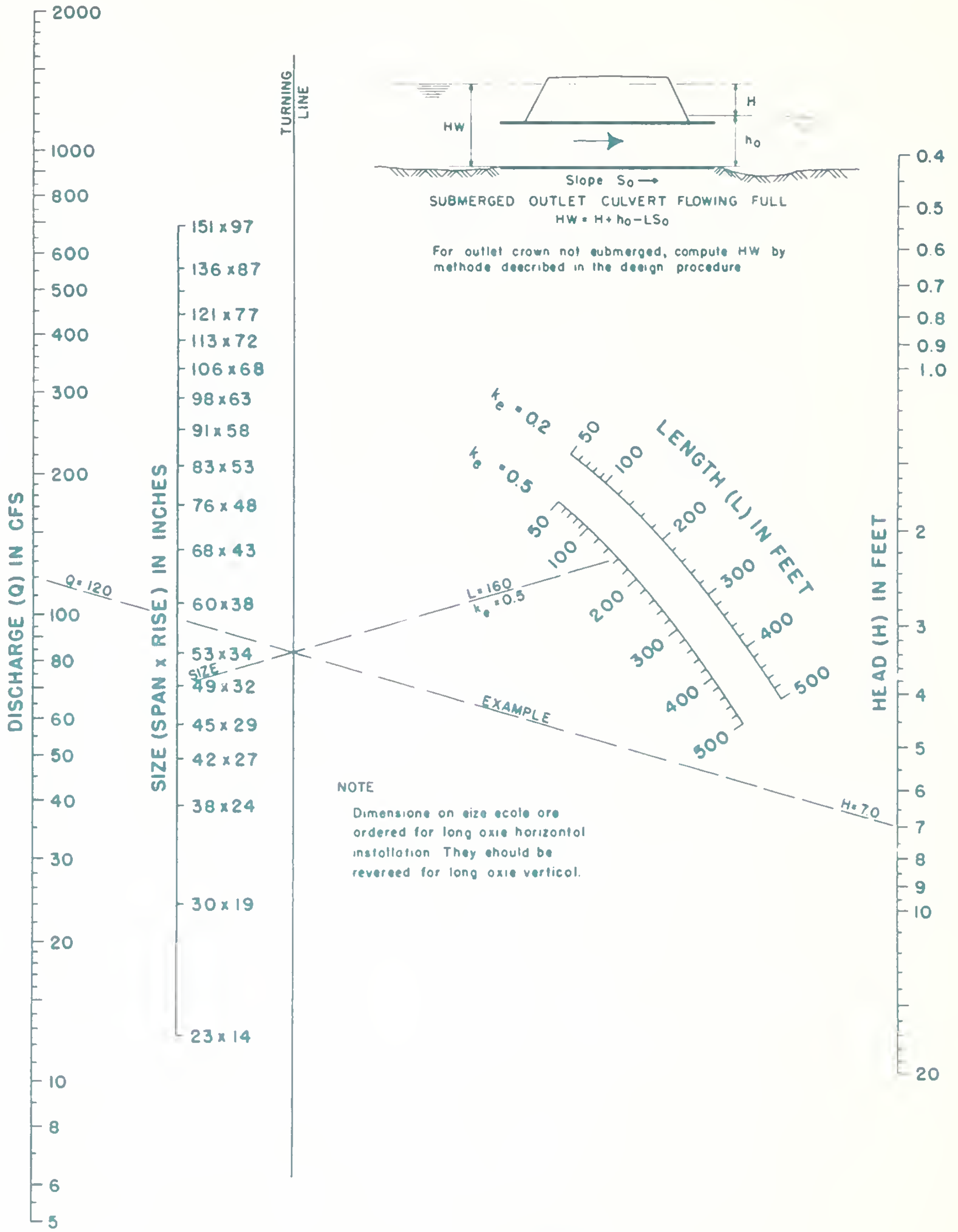

HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL

$n = 0.012$

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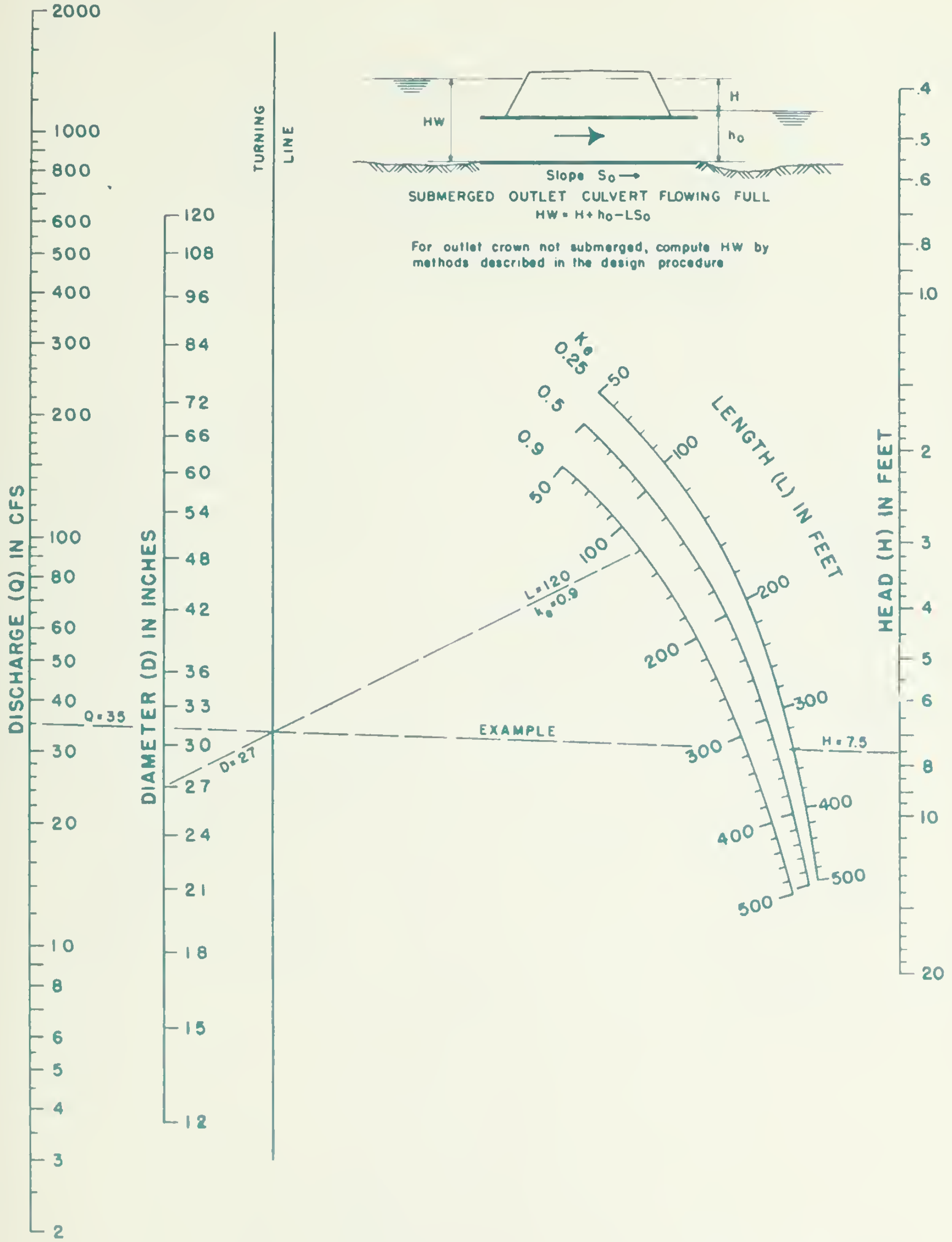
CHART 11



HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL
n = 0.024

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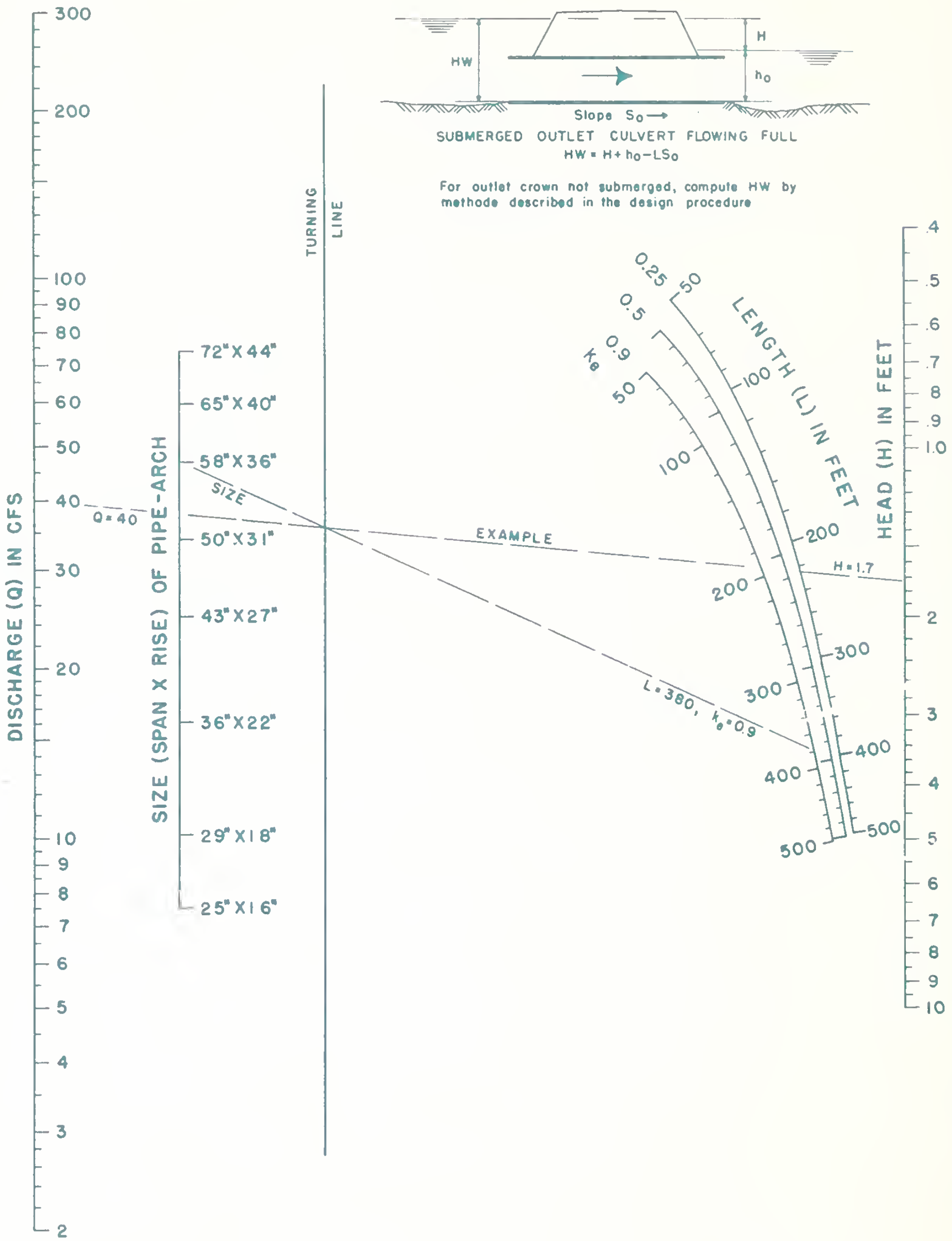

HEAD FOR STANDARD C. M. PIPE-ARCH CULVERTS FLOWING FULL
n=0.024

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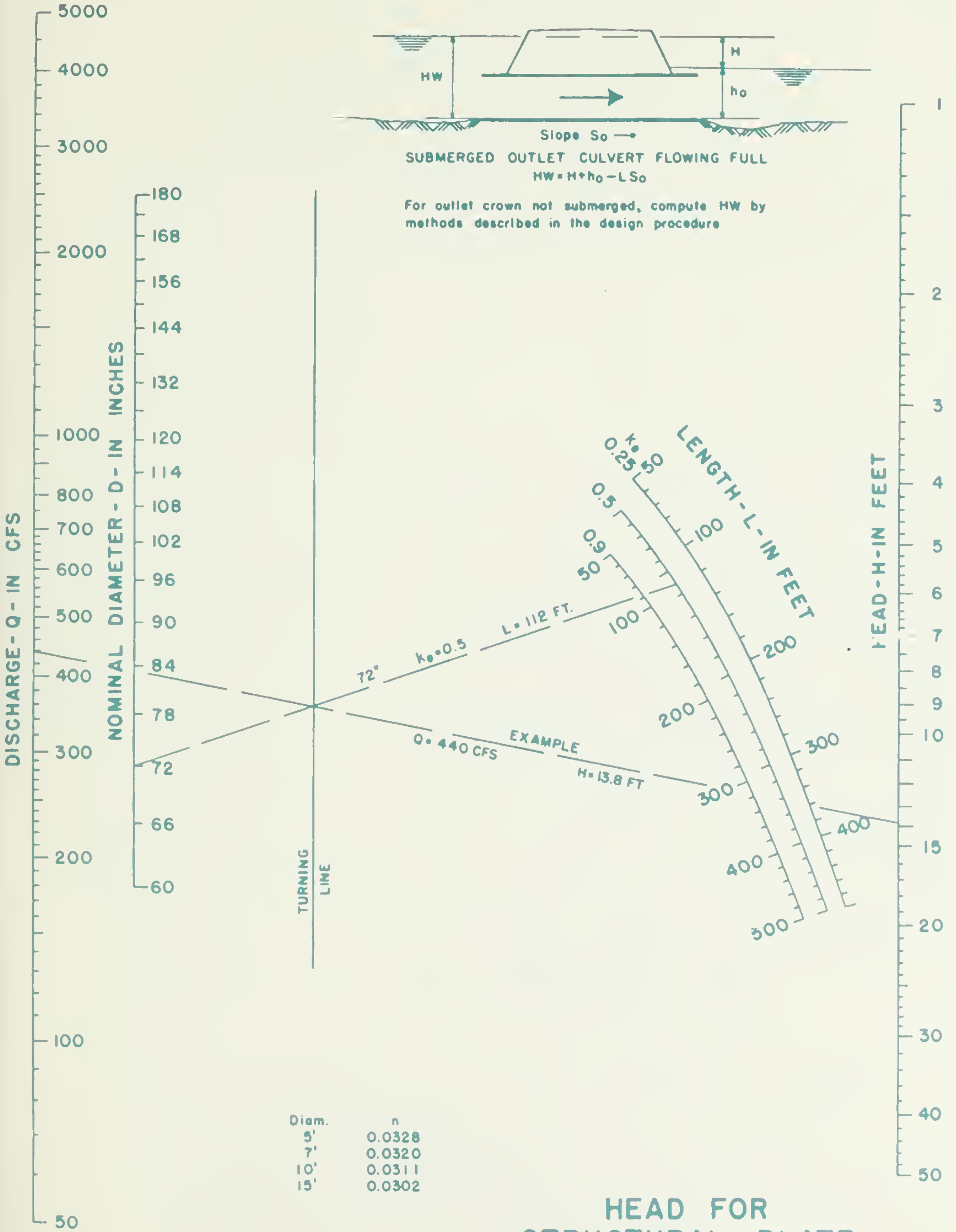

HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL

$n = 0.0328$ TO 0.0302

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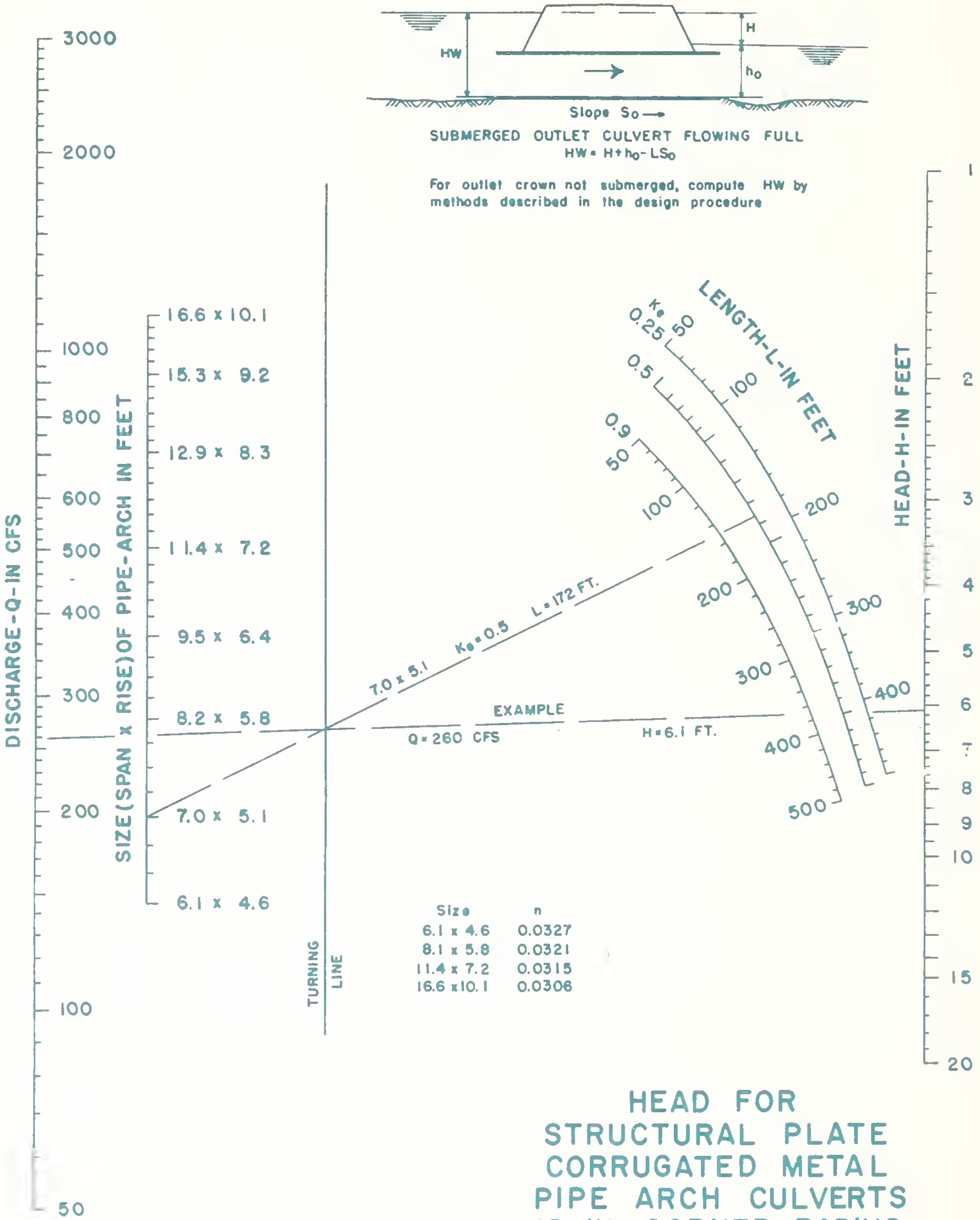

HEAD FOR
STRUCTURAL PLATE
CORRUGATED METAL
PIPE ARCH CULVERTS
18 IN. CORNER RADIUS
FLOWING FULL
n=0.0327 TO 0.0306

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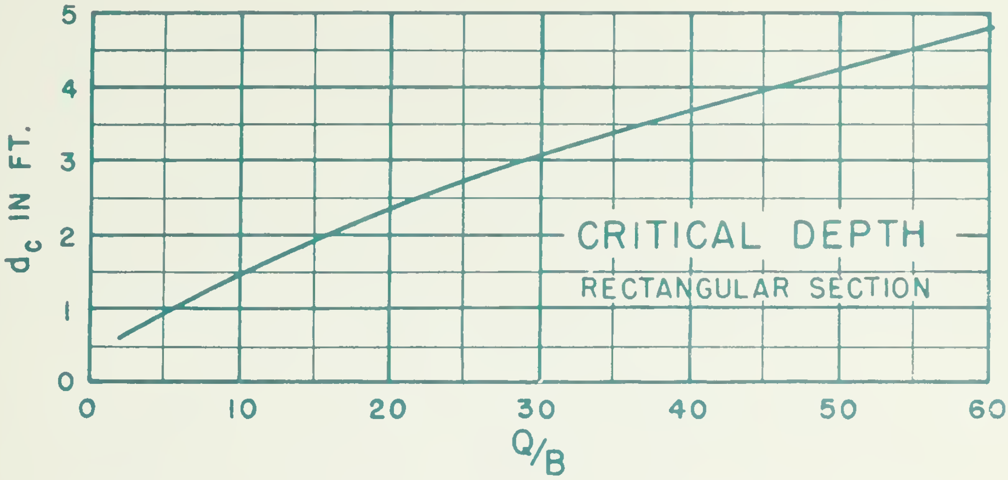

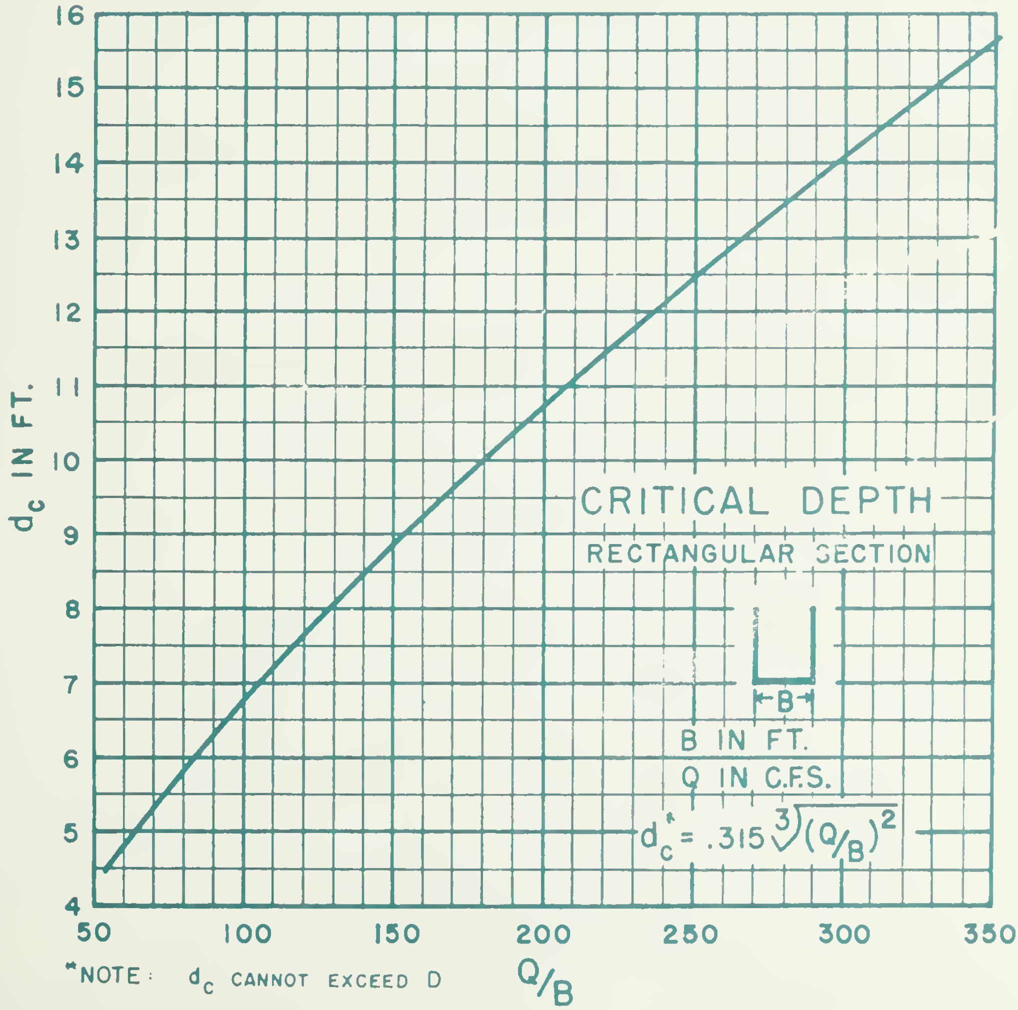

*NOTE: d_c CANNOT EXCEED D

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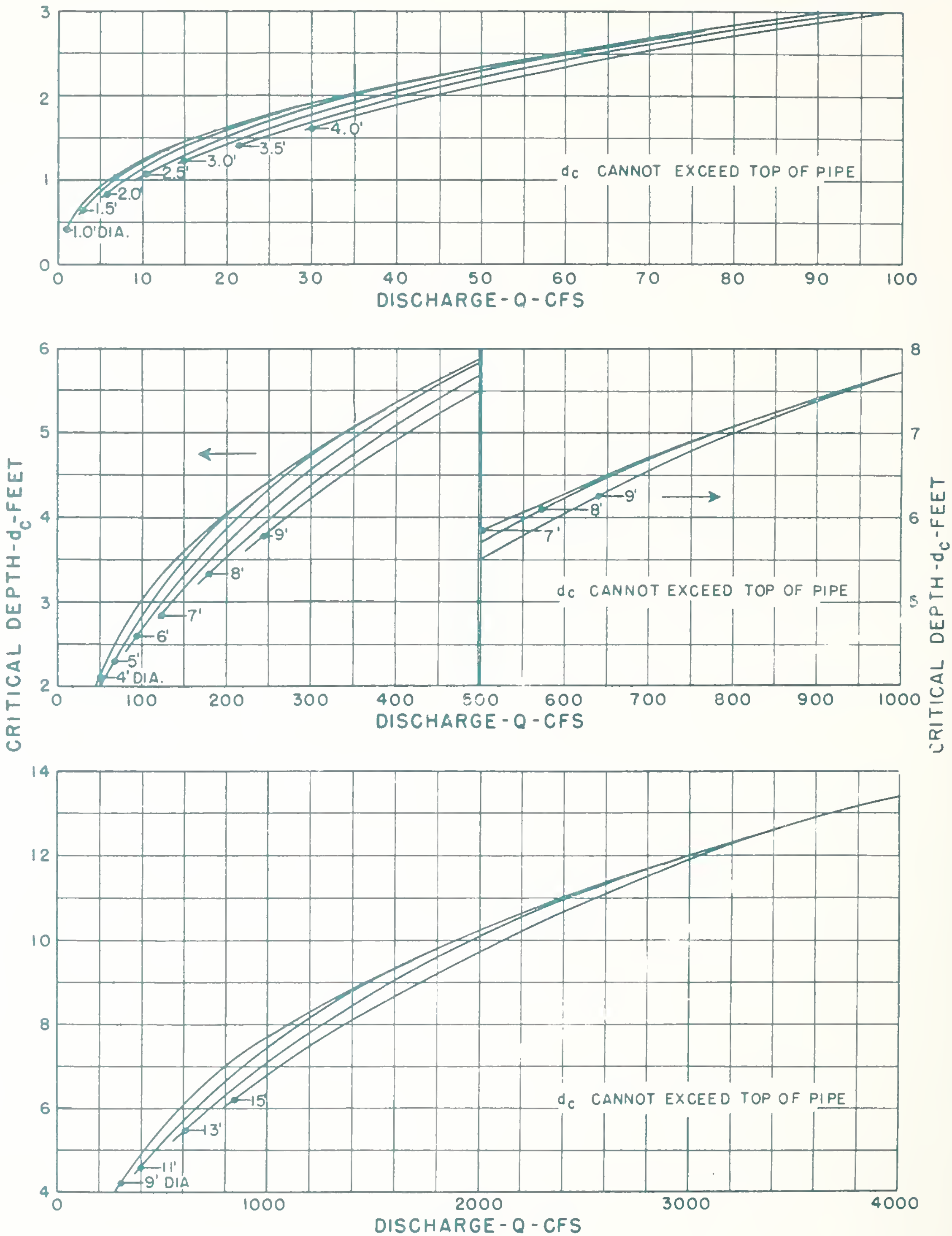
CHART 16



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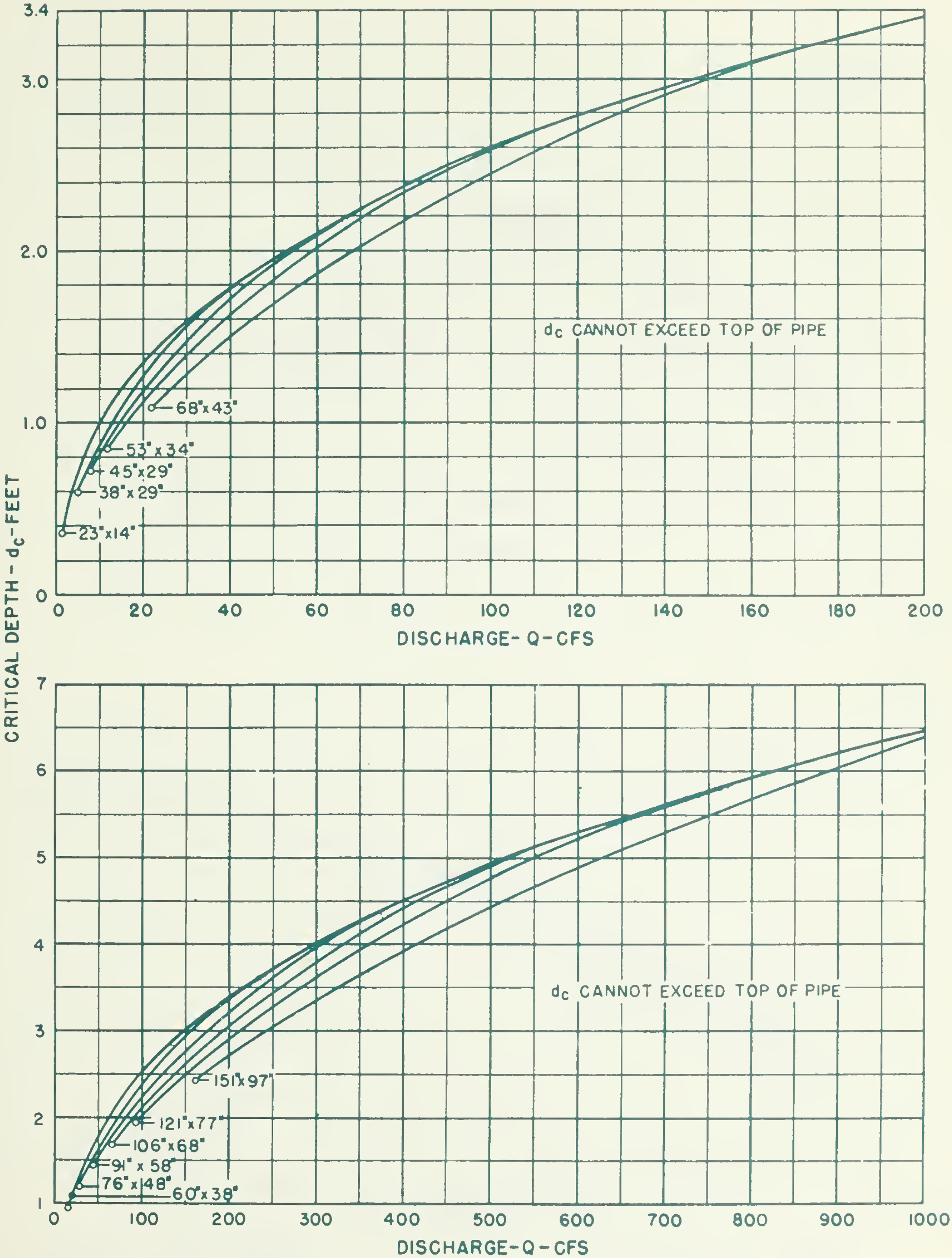
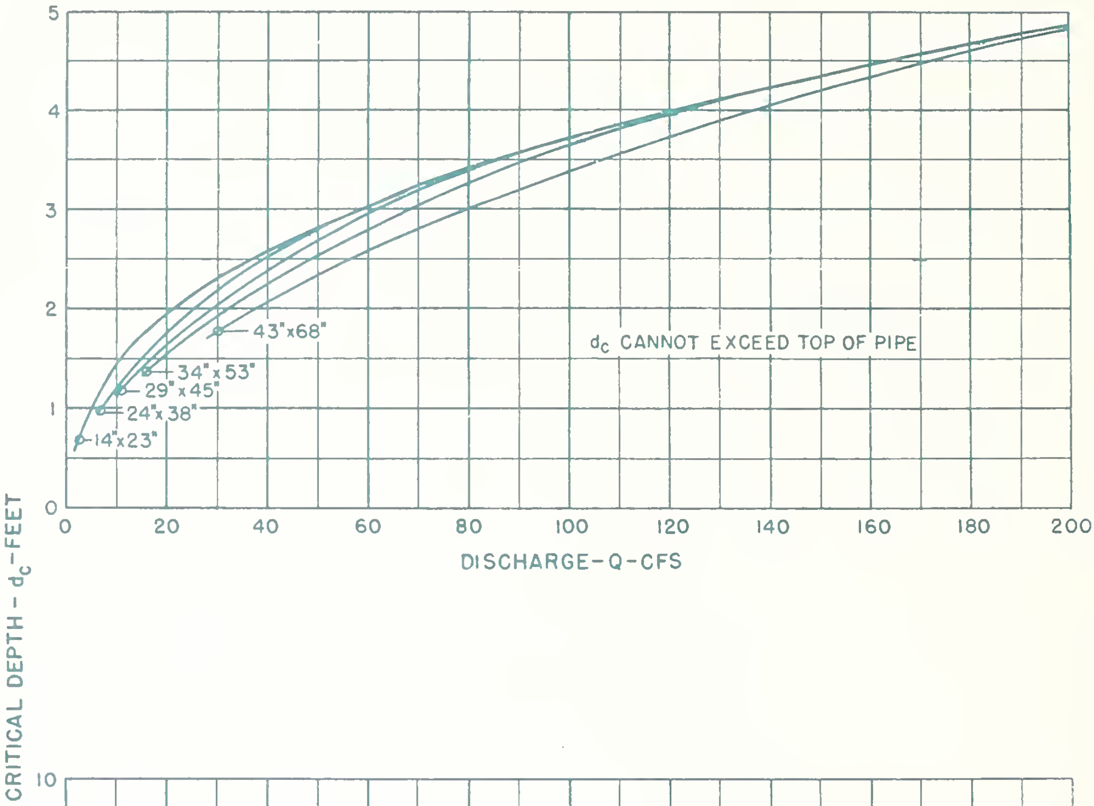
CRITICAL DEPTH
CIRCULAR PIPE

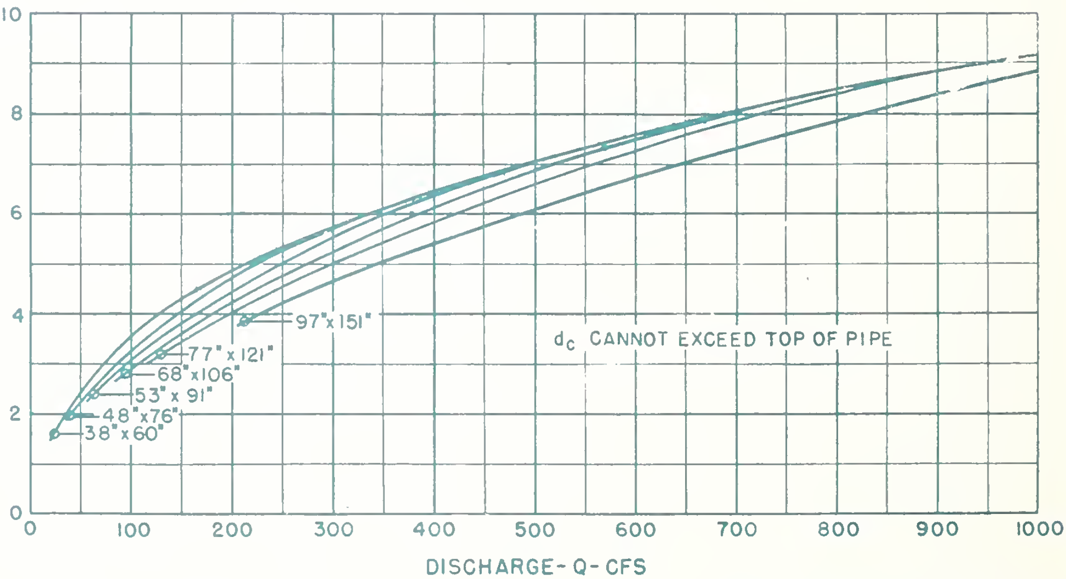


BUREAU OF PUBLIC ROADS

JAN. 1964

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL





BUREAU OF PUBLIC ROADS
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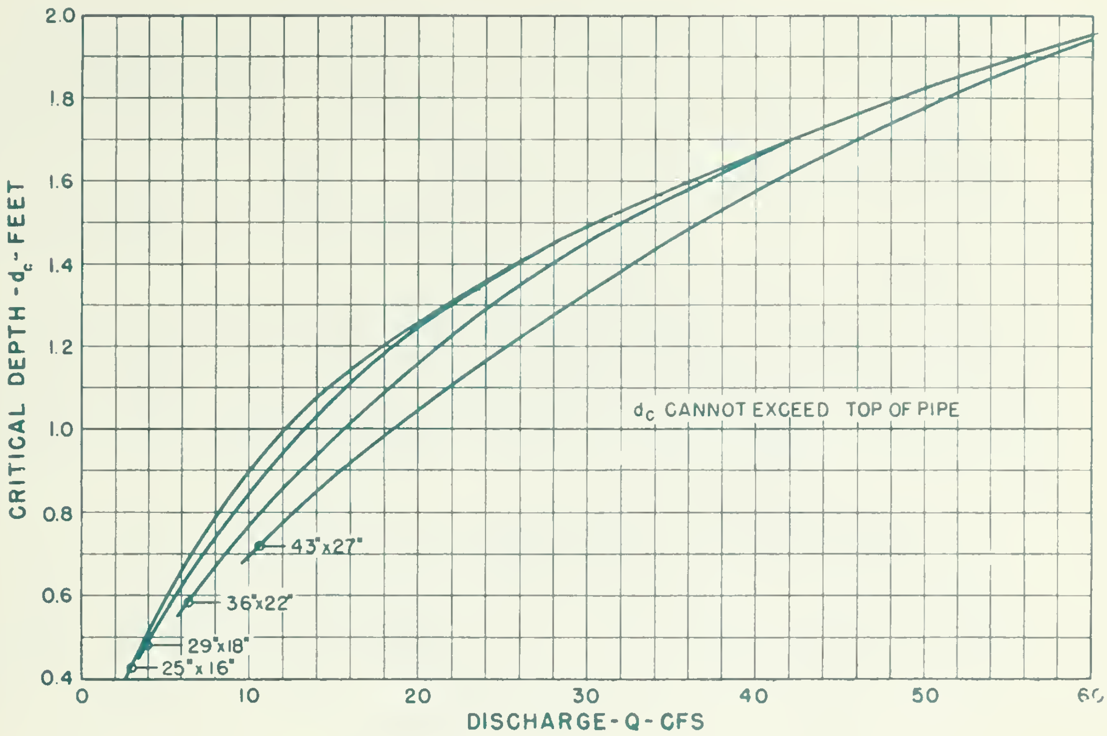
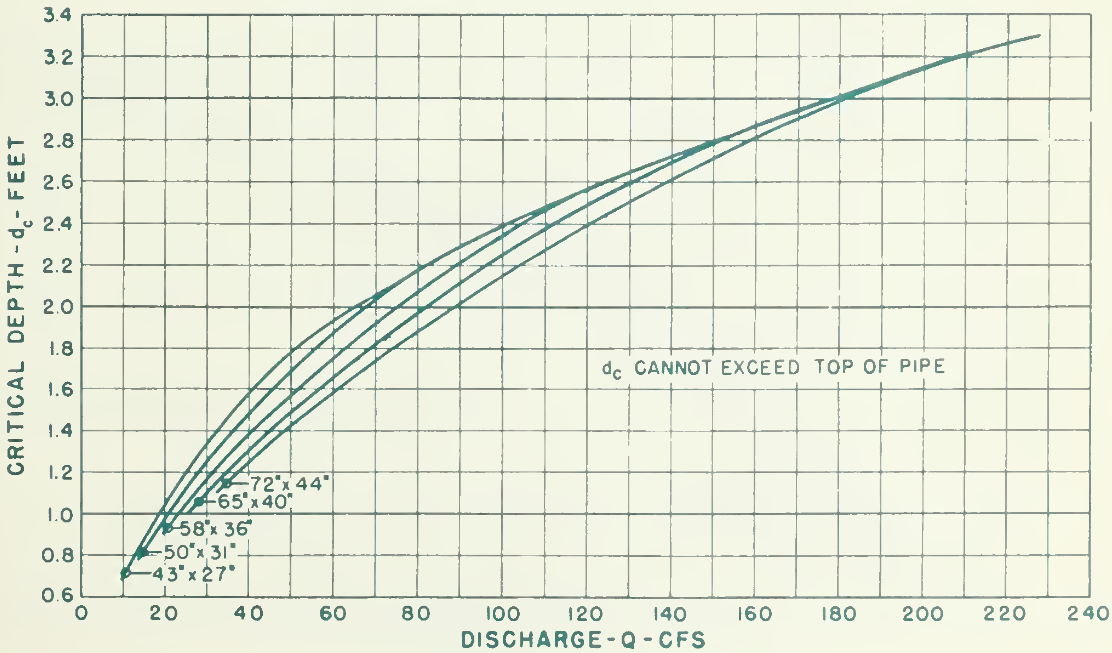
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL

CHART 19





BUREAU OF PUBLIC ROADS
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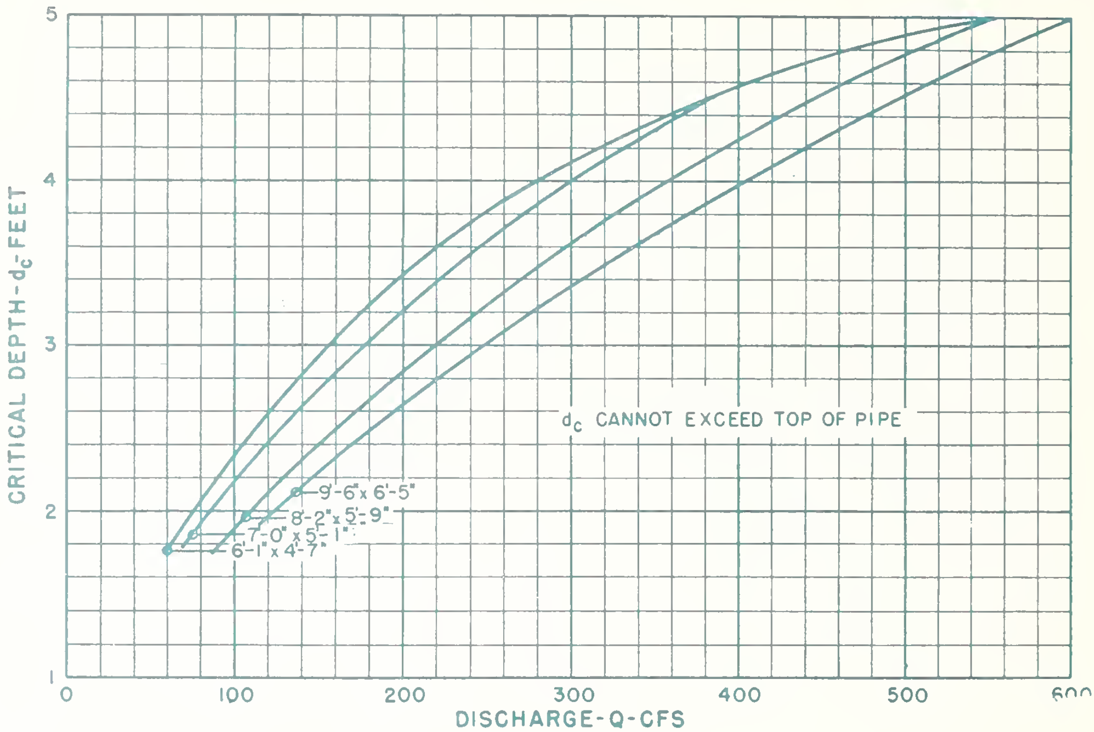
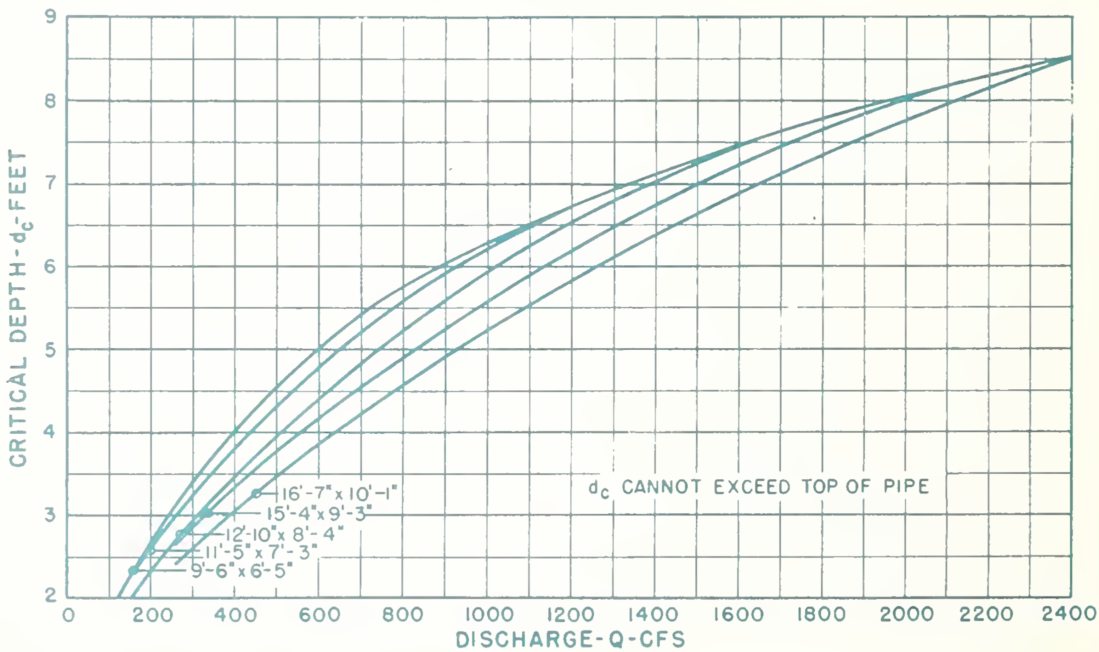
CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH

CHART 20





BUREAU OF PUBLIC ROADS

JAN. 1964

CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH

18 INCH CORNER RADIUS

Appendix A - PERFORMANCE CURVES

The principal disadvantage in using nomographs for the selection of culvert sizes is that it requires the trial and error solution described in this circular. Some engineers who limit their selection to a relatively small number of types of culverts would find it advantageous to prepare performance curves such as shown in figure 8. These curves are applicable through a range of headwaters and discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes.

Figure 8 is plotted from the data shown in the following tabulations. These data were obtained from the nomographs contained in this circular. (Computer programs are available from Public Roads for making these computations.) The first tabulation is for the inlet-control curve on figure 8, and the second tabultation is for the outlet-control curves.

Data for Inlet-Control Curve

$\frac{HW^*}{D}$ (Assume)	Q* (Read)	HW $\frac{HW}{D} \times 4$
.5	21 c.f.s.	2.0 ft.
.6	29	2.4
.7	37	2.8
.8	46	3.2
.9	56	3.6
1.0	65	4.0
1.1	74	4.4
1.3	90	5.2
1.5	102	6.0
1.7	112	6.8
2.0	126	8.0
2.5	145	10.0
3.0	165	12.0

*From Chart 5 Projecting Inlet (3)

Data for Outlet-Control Curves

Q	d_c	$\frac{d_c + D}{2}$	H	HW for Various S_o				
(Assume)	Chart 16	(Compute)	Chart 11	0%	.5%	1%	1.5%	2.0%
20 cfs	1.3 ft.	2.6 ft.	.2* ft.	2.8 ft.	-	-	-	-
40	1.9	3.0	.8	3.8	2.8	1.8	.8	-
60	2.3	3.2	1.9	5.1	4.1	3.1	2.1	1.1
80	2.7	3.4	3.3	6.7	5.7	4.7	3.7	2.7
100	3.1	3.6	5.2	8.8	7.8	6.8	5.8	4.8
120	3.3	3.6	7.5	11.1	10.1	9.1	8.1	7.1
140	3.5	3.8	10.2	14.0	13.0	12.0	11.0	10.0
160	3.7	3.8	13.6	17.4	16.4	15.4	14.4	13.4

$HW = H + h_o - LS_o$ where $h_o = \frac{d_c + D}{2}$

*From Chart 11 - or by Equation 2.

The curves plotted apply only to the type and length of culvert shown. Culverts placed on grades steeper than about 2.5 percent will operate on the inlet control curve for the headwater-discharge range of this plot. If a free outfall condition does not exist a correction for tailwater should be made as instructed in Step 3b, p. 5-16 of "Procedure for Selection of Culvert Size".

HYDRAULIC PERFORMANCE CURVES
FOR
48-INCH C.M. PIPE CULVERT
WITH
PROJECTING INLET

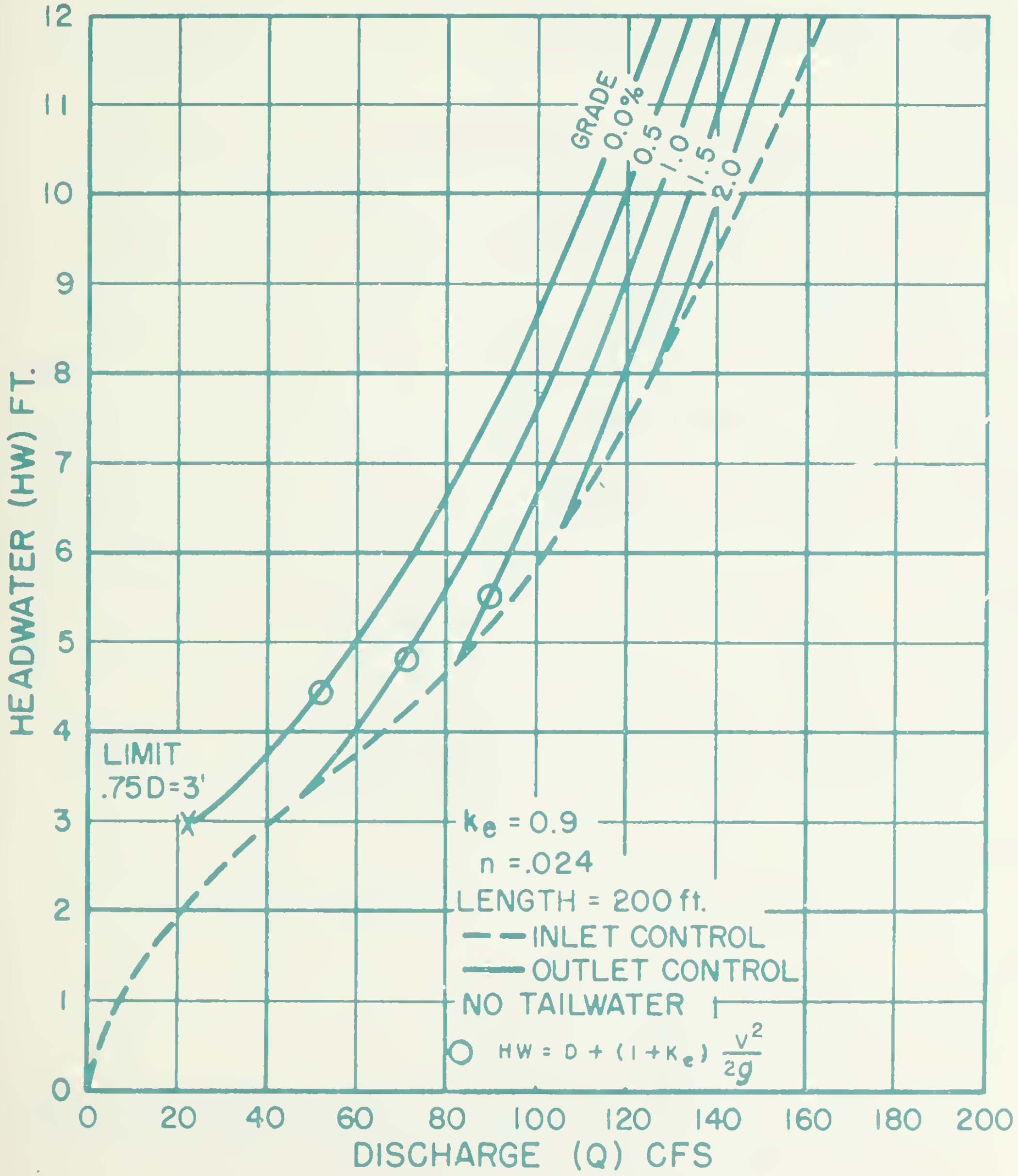


Figure 8

Table 1. - Entrance Loss Coefficients

Coefficient k_e to apply to velocity head $\frac{V^2}{2g}$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operating full or partly full with control at the outlet.

$$\text{Entrance head loss } H_e = k_e \frac{V^2}{2g}$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square-edge	0.5
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7

*Note: "End Section conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the bevelled inlet, p. 5-13.

Table 2. - Manning's n for Natural Stream Channels[5]
(Surface width at flood stage less than 100 ft.)

1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030--0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035--0.05
c. Some weeds, light brush on banks	0.035--0.05
d. Some weeds, heavy brush on banks	0.05 --0.07
e. Some weeds, dense willows on banks	0.06 --0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01 --0.02
2. Irregular sections, with pools, slight channel meander; <u>increase</u> values given above about	0.01 --0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04 --0.05
b. Bottom of cobbles, with large boulders	0.05 --0.07

[5] From "Design Charts for Open Channel Flow", (see p. 5-14).

PROJECT: I-46 (1)

DESIGNER: J. A. F.

DATE: 2-18-64

HYDROLOGIC AND CHANNEL INFORMATION

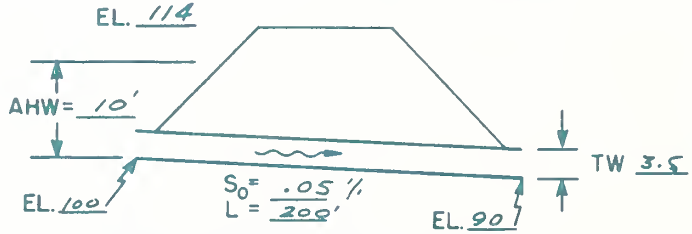
Q_1 = 180 cfs. = Q_{25} TW_1 = 3.5

Q_2 = 225 cfs. = Q_{50} TW_2 = 4.0

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 6+21



MEAN STREAM VELOCITY = 10'/sec

MAX. STREAM VELOCITY = 14'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. HW/D	INLET CONT. HW	OUTLET CONTROL $HW=H+h_o-LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CIRCULAR CMP PROJ. ENT.	180		ASSUME 1.5	7.5	D = 60"+ try smaller size AHW = 10'											
"	180	54"	2.2	9.9	.9	9.7	3.9	4.2	3.5	4.2	10.0	3.9	9.9	16.5		
"	225	54"	3.15	14.2	.9	15.3	4.2	4.4	4.0	4.4	10.0	9.7	14.2	17.0		HW high for Q_{50} - Try 60"
"	180	60"	1.51	7.55	.9	5.9	3.9	4.4	3.5	4.4	10.0	0.3	7.55	16.7		
"	225	60"	2.1	10.5	.9	9.3	4.2	4.6	4.0	4.6	10.0	3.9	10.5	17.5		

SUMMARY & RECOMMENDATIONS: Velocities read from char 46, 47 - "Design Charts for Open Channel Flow". (See p. 5-14). Outlet velocities are about the same for each size, indicating change in size has little effect. Size selected (60 or 54-inch) depends on designer's confidence in flood estimate and damage incurred if a larger flood should occur. Note that TW must be greater than 10.1' for outlet control to govern for the 54" pipe flowing 180 cfs. Accurate determination of TW depths is unnecessary in most cases.

PROJECT: 142 B

DESIGNER: L.O.K

DATE: 2-18-64

HYDROLOGIC AND CHANNEL INFORMATION

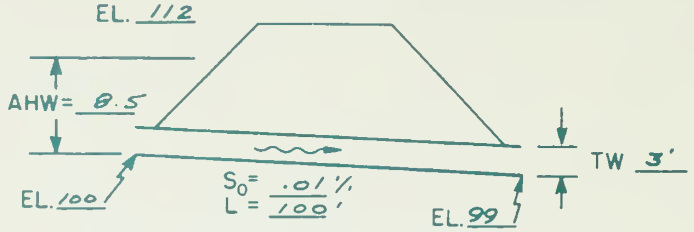
Q_1 = 160 cfs = Q_{50} TW_1 = 3.0'

Q_2 = ____ TW_2 = ____

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 321+14



MEAN STREAM VELOCITY = 8'/sec

MAX. STREAM VELOCITY = 10'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. HW/D	HW	OUTLET CONTROL HW=H + h_o − LS_o: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CMP (cir.) Headwall	160	Assume 54"	1.56	7.0												Hw less than 8.5' - Try 48"
"	160	48"	2.25	9.0	.5	8.3	3.7	3.8	3	3.8	1.0	11.1	11.1	13.2'/sec		Hw High Try 54"
"	160	54"	1.56	7.0	.5	4.7	3.6	4.1	3	4.1	1.0	7.8	7.8	11.1'/sec		velocity at d_c size O.K.
Concrete (cir) Sq. Edge - Hdwl	160	48"	2.35	9.4	.5	4.7	3.7	3.8	3	3.8	1.0	7.5	9.4	14'/sec		Hw high Try 54"
"	160	54"	1.6	7.2	.5	2.9	3.6	4.1	3	4.1	1.0	6.0	7.2	14.7'/sec		Hw ok. [illegible] > CMP Try 48 G. Edge
Concrete (cir) Groove end - Hdwl	160	48	1.95	7.8	.2	4.0	3.7	3.8	3	3.8	1.0	6.8	7.8	14.0'/sec		Hw ok Vel. high

SUMMARY & RECOMMENDATIONS:

The selection of a 54" CMP with headwall will keep the headwater below the AHW with a minimum outlet velocity. A 48" concrete pipe with groove edged entrance gives equal HW and slightly higher outlet velocity. Protection of outlet channel might be necessary in some locations.

PROJECT: E 14-2 (3)

DESIGNER: F.P.R.

DATE: 2-20-64

HYDROLOGIC AND CHANNEL INFORMATION

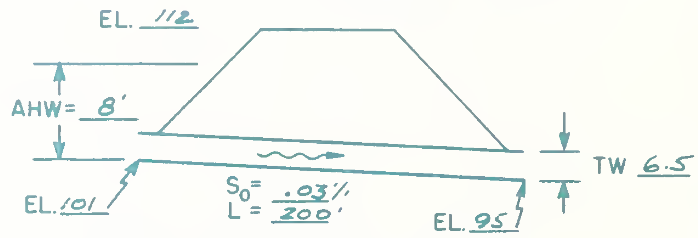
Q_1 = 400 cfs. Q_{50} TW_1 = 6.5'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 8+61



MEAN STREAM VELOCITY = 8'/sec

MAX. STREAM VELOCITY = 12'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW=H+h_o-LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
Concrete (CIR) Gr. End Proj	400		Assume 1.5			Find	D =	78"								HW = 9.1 Too high-Try 84"
"	400	84"	1.18	8.3												HW High Try 90"
"	400	90"	1.05	7.9	.2	1.9	5.2	6.3	6.5	6.5	6.0	2.4	7.9	28'/sec I.C. 10'/sec O.C.		If too large Try 2 pipes
Same type 2 pipes	200	54"	1.85	8.3												Too small
"	200	60"	1.38	6.9	.2	3.4	4.0	4.5	6.5	6.5	6.0	3.9	6.9	23'/sec I.C. 10'/sec O.C.		Use See comments
Cir. CMP Bevel B (chart 7)	200	60"	1.34	6.7	.25	6.2	4.0	4.5	6.5	6.5	6.0	6.7	6.7	14'/sec I.C. 10'/sec O.C.		Use. Bevel A can be used here
		Might try single concrete oval or metal arches														

SUMMARY & RECOMMENDATIONS: Problem to illustrate use of double pipes if one pipe is too high or not available. Inlet control governs. TW submerges culvert outlet for all double barrels. Velocities are computed for both inlet control and for full flow at outlet caused by TW. Two 60-inch concrete pipes or two 60-inch CMP with inlets shown satisfy headwater limitations. Concrete pipe will give considerably higher outlet velocities if tailwater is not effective in causing the culvert to fill at the outlet.

PROJECT: I 85-2

DESIGNER: L. A. H.

DATE: 2-23-64

HYDROLOGIC AND CHANNEL INFORMATION

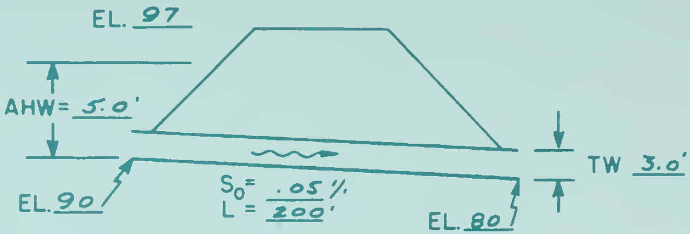
Q_1 = 120 cfs. = Q_{25} TW$_1$ = 3.0'

Q_2 = ______ TW$_2$ = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 314 + 10



MEAN STREAM VELOCITY = 12'/sec
MAX. STREAM VELOCITY = 15'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. HW/D	HW	OUTLET CONTROL HW=H + h_o − LS_o: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CMP (Cir.) Mitered	120	Assume 54"	1.25	5.6'								outlet control does not govern		Not computed - rock channel		Hw high Try 60"
"	120	60"	.97	4.9	.7	2.5	3.0	4.0	3.0	4.0	10.0		4.9			Need more cover - Try arch
CMP Arch Mitered	120	72" x 44"	1.24	4.6	.7	3.4	2.4	3.0	3.0	3.0	10.0		4.6			check box Culvert
Concrete Box 30° w.w.	120	4' x 4'	1.23	4.9	.4	2.0	3.1	3.5	3.0	3.5	10.0		4.9			
Concrete Oval Gr. End Proj.	120	60" x 38"	1.51	4.8	.2	2.9	2.7	2.9	3.0	3.0	10.0		4.8			
Concrete Cir Groove End Proj	120	54"	1.11	5.0	.2	1.7	3.1	3.8	3.0	3.8	10.0		5.0			

SUMMARY & RECOMMENDATIONS:

IN-PLACE COST, AVAILABILITY, LOCATION, COVER REQUIREMENTS, ETC., SHOULD BE CONSIDERED BY THE DESIGNER IN SELECTING CULVERT. CM PIPE ARCH CULVERTS OR CONCRETE OVAL PIPES MIGHT BE A SOLUTION WHERE COVER IS LIMITED.

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